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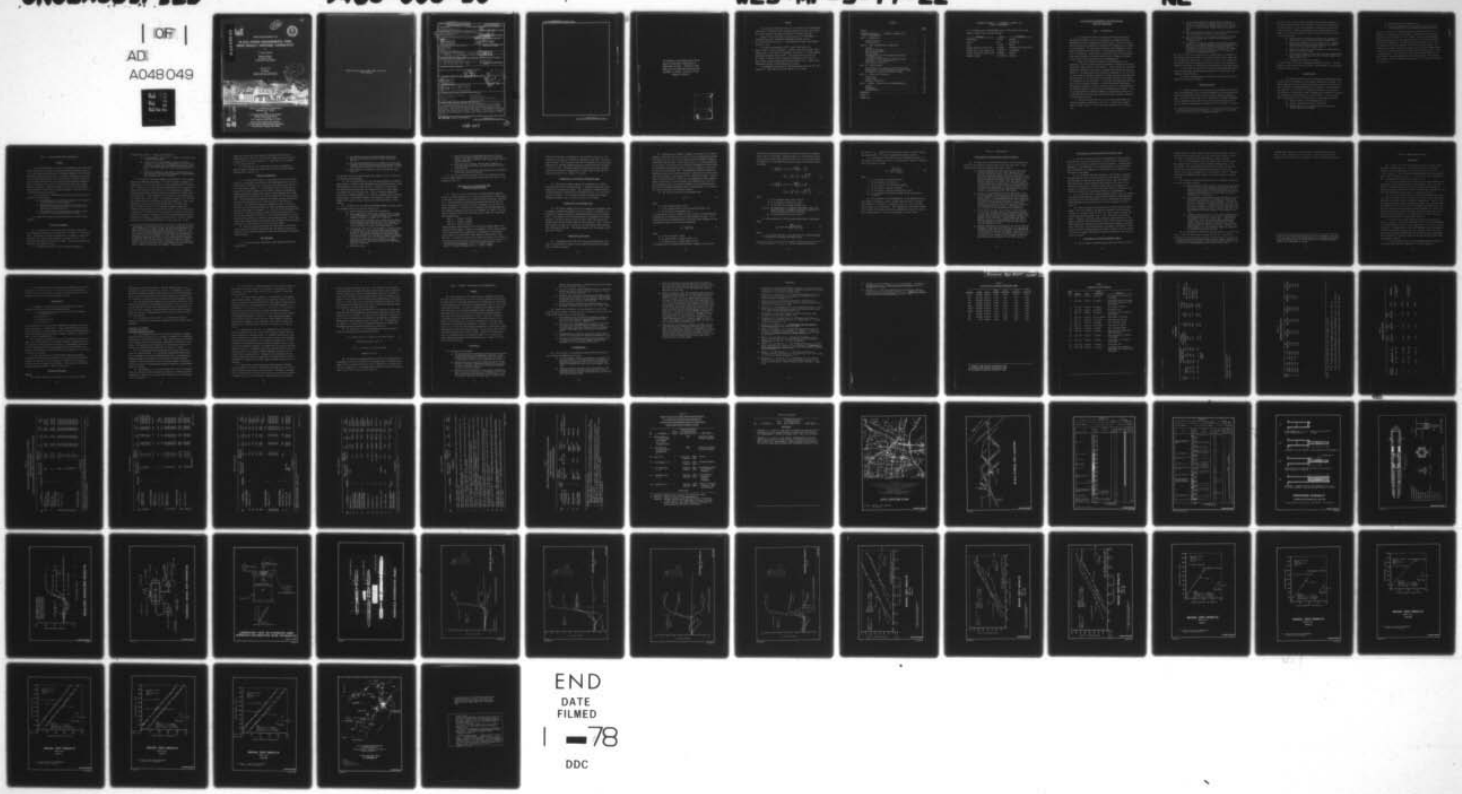

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IN SITU STRESS MEASUREMENTS, PARK RIVER PROJECT, HARTFORD, CONNECTICUT

by

Dr. Mysore Nataraja

Dames and Moore
6 Commerce Drive
Cranford, New Jersey 07016

November 1977

Final Report

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This report presents the test procedure and results of in situ stress tests performed along the proposed route of the Park River Tunnel, Hartford, Connecticut. The borehole deformation gage overcoring technique was successfully used to depths of 150 ft below ground surface. Modifications to the test procedure and equipment are discussed. The results of these tests are compared with the stresses determined at several other sites in the northeastern United States.

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PREFACE

This report contains the results of a contract field investigation (Contract No. DACW39-77-C-0037) by Dames & Moore, Cranford, New Jersey. The project engineer for Dames & Moore was Dr. Mysore Nataraja.

Funds for this study were provided by the U. S. Department of Transportation under Reimbursable Agreement Document Control No. DOT-AS-70014 and by the U. S. Army Engineer Division, New England, IAO No. 77 C-24.

The study was performed in FY 77 under the direction of Messrs. James P. Sale and Richard G. Ahlvin, Chief and Assistant Chief, respectively, of the Soils and Pavements Laboratory, U. S. Army Engineer Waterways Experiment Station (WES). The contract was monitored by Mr. Robert D. Bennett under the general supervision of Mr. Jerry S. Huie, Chief, Design Investigations Branch, and Mr. Don C. Banks, Chief, Engineering Geology and Rock Mechanics Division.

The Commander and Director of WES during this study was COL John L. Cannon, CE. The Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
microinches	0.0254	microns
inches	25.4	millimetres
feet	0.3048	metres
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per square foot	47.88026	pascals
degrees (angle)	0.01745329	radians

IN SITU STRESS MEASUREMENTS, PARK RIVER PROJECT, HARTFORD, CONNECTICUT

PART I: INTRODUCTION

1. To evaluate the stress concentration effects caused by an underground opening in rock, it is necessary to estimate the three-dimensional in situ or preexcavation stress field. The stress concentrations acting on an excavation influence the deformation of the walls of the excavation. In situ tests, therefore, are necessary, to determine the magnitude and orientation of the principal stresses in rock prior to excavation. Models used in laboratory tests do not and generally cannot simulate the geologic factors such as bedding, fractures, joints, faults, folds, and macroscopic inhomogeneities.

2. Among the procedures available for determining in situ stresses in rocks the overcoring method attempts to determine the three-dimensional stress field in one operation. In this method the diametral changes are measured in the plane of deformation, during the overcoring operation. By making simplified assumptions regarding the in situ state of stress, and by applying equations of linear elasticity, the magnitude and directions of the major and minor principal stresses in the plane of deformation can be determined. To determine the magnitudes and directions of the principal stresses, (a) three deformation measurements of the diametral changes of a hole in the rock and (b) the determination of the elastic modulus of the rock under consideration are required. This in situ measurement of diametral changes is accomplished by the overcoring procedure with the rock modulus determined by biaxial testing of the extracted rock core.

3. This report describes the in situ stress measurements performed at Hartford, Connecticut, for the U. S. Army Corps of Engineers by Dames & Moore. Some of the unique features of this project are:

a. In situ stress tests were successfully performed to a depth of 155 feet* using standard overcoring procedures (originally developed for shallow depths) using the standard U. S. Bureau of Mines (USBM) borehole deformation gage;

b. Some modifications were made to the standard USBM gage and their advantages and limitations were studied;

c. Some modifications to the drilling procedures and equipment were made and their effects were evaluated and documented; and

d. A comparative study was attempted to evaluate the performances of the standard USBM gage, modified USBM gage, and three-axis borehole deformation gage manufactured by Terrametrics of Golden, Colorado. (However, the comparative study of the three gages could not be completed because of equipment malfunction.)

The site of the in situ stress tests is located along the alignment of the proposed auxiliary conduit of the water diversion tunnel under the City of Hartford, Connecticut. The results of the in situ stress tests are presented and their compatibility with the rest of the available regional stress data for northeastern North America is discussed briefly.

4. The "pros" and "cons" of the borehole deformation gage modification and equipment and procedures modification for deep hole overcoring are also presented.

5. Finally, recommendations are made for any future work in the deep hole overcoring procedures.

Purpose and Scope

6. The main purpose of this project was to demonstrate the feasibility of performing in situ stress measurements by the overcoring technique at depths greater than 100 feet, using a borehole deformation gage. The United States Army Corps of Engineers is presently undertaking the design and construction of a water diversion tunnel under the City of Hartford, Connecticut (at depths of 150 to 200 feet). The site

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

for the in situ stress test was intended to be along the alignment of the water diversion tunnel with the depth of the test corresponding to the tunnel depth, so that the information obtained from this project could be used for checking the adequacy of the lining design of the tunnel. In order to accomplish the above purpose, the following scope of work was defined:

- a. Review existing overcoring procedures and equipment and modify them to suit the needs of deep borehole tests;
- b. Review the capabilities and limitations of the standard USBM borehole deformation gage, and modify the gage to suit the needs of deep borehole tests;
- c. Review the literature on and evaluate any other existing borehole deformation gages;
- d. Perform deep borehole in situ tests with the standard USBM gage, modified USBM gage, and other available gages which seem suitable; and
- e. Interpret and discuss test results.

The above scope of work was accomplished during this project. The procedures, results, and discussions thereof are presented in the following sections of this report.

Site Selection

7. All existing information on the exploratory borings previously drilled along the alignment of the auxiliary conduit was reviewed. The boring logs and the geologic profile along the central line of the conduit, presented in Design Memorandum No. 9 prepared by the Department of the Army (Ref. 1), were used to establish prospective test locations. The location of the auxiliary conduit alignment with respect to surrounding natural and cultural features is shown on Plate 1. The criteria used in selecting the site to perform the in situ stress tests were:

- a. Ease of access to truck-mounted drill rig;
- b. Minimum overburden at the test location;
- c. Minimum water loss in pressure tests in previously drilled exploration borings;

d. High RQD and percent recovery; and

e. The need to perform the in situ stress tests at or near a section which will be instrumented during tunnelling.

Several locations were eliminated because the above criteria were not satisfied. A location close to Boring No. FD-25T (Ref. 1) was considered ideal from all points of view. However, this site was submerged under the flood waters of the Connecticut River at the time of initiating the field operations. Therefore, the second choice, a site close to Boring No. FD-30T, was selected for the in situ stress tests. The chosen site is located inside the parking lot of the Good Shepard Church in the City of Hartford and was considered ideal from the point of view of accessibility. The coordinates of the test location identified as OC-1 (Plate 2) are approximately N148,340 and E170,395. The thickness of overburden (which is predominantly glacial till) is less than 60 feet at this site.

PART II: TEST PROCEDURES AND COMPUTATIONS

General

8. The determination of in situ rock stresses by overcoring techniques consists of inserting a three-axis borehole deformation gage in a 1-1/2-inch borehole and measuring the diametral changes of this borehole during overcoring (stress relief). The process essentially consists of (a) drilling a small-diameter hole within and for a distance beyond the end of a large-diameter hole, (b) positioning the borehole deformation gage in the smaller hole, and (c) coring over the gage with a large-diameter diamond bit. Stresses acting on the rock are relieved upon overcoring and are reflected in diametral changes of the rock core as it is separated from the rock mass.

9. The procedure of determining the in situ stress can be subdivided into three phases:

- a. The measurement of the diametral changes of the borehole during overcoring (stress relief);
- b. The determination of the modulus of elasticity of the rock core in the laboratory or at the site by recompression in a biaxial cell; and
- c. The computation of stresses using the theory of linear elasticity, measured deformations, and moduli.

10. These three phases are described briefly in the following sections.

Field Test Procedure

11. A 7-inch-O.D. tri-cone roller bit was used to drill hole OC-1 through the overburden to a depth of 61' 3" (about 5' 9" into bedrock). The borehole was then enlarged with a 13-inch-O.D. roller bit, and 8-inch-I.D. steel casing installed to a depth of 61' 3". The casing was then grouted in place with cement. The boring log for borehole OC-1 is presented in Plate 3.

12. The general concepts of the overcoring technique are

illustrated in Plate 4. Simply, the procedure is:

<u>a</u>. A large-diameter (6-inch-O.D.) borehole is drilled to the desired testing depth;

<u>b</u>. A concentric, 1.5-inch-diameter borehole ("EX" plus reamer) is then drilled to approximately 24 inches below the level at which the large-diameter borehole is terminated. The "EX" borehole is then scribed* along axis I; and

<u>c</u>. A borehole deformation gage (BDG) is inserted into the "EX" hole and "overcored" by extending the large-diameter hole an additional 12 to 18 inches.

13. During the overcoring operation, using water as the drilling medium, the diametral deformation of the "EX" borehole is measured on three axes (I, II, III) 60° apart in a plane perpendicular to the borehole. Deflections of cantilever arms (contained within the gage and in contact with the wall of the "EX" hole) are transmitted as changes in electrical resistivity through transducers attached to the upper ends of the cantilevers. These transducers are connected to the strain indicator read-out device via a cable. The diametral changes of the "EX" hole are read at every 1/2-inch advance of the overcoring run. Three strain indicators are used to record the deformations of the three axes. Plate 5 shows the details of the borehole deformation gage and a schematic representation of the field test setup. Once the overcoring is completed, the rotation of the core barrel is stopped, but the flow of water is allowed to continue in order to minimize thermal change and stabilize the readings. Readings are continued for a period of time to account for any time-dependent behavior of the rock core. At the

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* The purpose of scribing the hole is to be able to identify the absolute orientation of U_1 axis, so that, during the determination of the elastic modulus in a biaxial chamber, the BDG can be placed in the "EX" hole in the same orientation as during the in situ test. However, during this field program it was realized that the presence of fractures in the rock could also be 'felt' while scribing the hole. By carefully examining the "EX" core and correlating the information obtained by this "feel technique," it was often possible to make valuable decisions, whether or not to perform a test at a particular depth. Such decisions were often crucial because of poor rock conditions frequently encountered and budgetary considerations.

completion of each test, the recorded deformation of each axis is plotted against the distance overcored and compared with an ideal plot to judge if the test was successful. An ideal plot for an overcore test is shown in Plate 6.

14. The basis for this overcoring procedure is described in greater detail by Hooker and Bickel (Ref. 2) and in the ASTM Special Publication No. 429 (Ref. 3).

Modulus Determination

15. To determine the modulus of elasticity of the stress relief cores, it is necessary to remove the 5.17-inch-diameter rock core (obtained from the 6.06-inch-O.D. core barrel) from the hole and reload it in a rock modulus chamber. For this project, the rock cores were inserted into a biaxial chamber, and the moduli values were determined. Loading was applied hydraulically (using a rubber membrane and a hand-operated hydraulic system) to the external surface of the stress relief core. The corresponding deformation of the small borehole was measured with the BDG reinstalled in its former position in the "EX" hole (Plate 7). The hydraulic pressure was applied in steps of 200 up to 2000 psi, and then decreased similarly, while measuring the deformation along the three axes of the gage (I, II, and III) for each pressure step. Two cycles of stress application of 200 to 2000 psi and then back to 200 psi were performed for each core. The recorded deformation of each axis was then plotted as a function of the applied pressure. From these curves, the average modulus of elasticity was calculated from the unloading curve at a strain level close to the in situ strain.

16. This test procedure requires an intact piece of overcored rock at least 10-1/2 inches long, which may not always be possible to obtain due to the natural fractures of the rock and mechanical breakage during drilling.

Test Equipment

17. The three borehole deformation gages employed during this program were:

a. The standard borehole deformation gage developed by USBM and later modified in 1974 (Hooker and Bickel, Ref. 2);

b. The modified USBM gage with the extended nose and rubber membrane covering the cantilever and transducers (certain modifications are similar to those found in Ref. 4); and

c. The Terrametrics three-axis borehole deformation gage (Ref. 5).

Item (a) was used as the primary gage while items (b) and (c) were used as secondary or backup gages.

18. Certain modifications to the drilling procedures were made by Dames & Moore. Instead of the recommended thin wall masonry bits, standard diamond drill bits (6.06-inch O.D., 5.19-inch I.D.) were used. Past experience indicates that the standard drill bit is more rugged and durable than the thin-wall bits. The core obtained during this project had a nominal diameter of 5.17 inches, slightly smaller than the USBM standard. The smaller diameter cores required a smaller biaxial chamber (for modulus testing), which was obtained specially for this purpose.

19. In addition to the borehole deformation gages, the other testing equipment used in the project consisted of:

a. Three Vishay Model P-350A Strain Indicators;

b. One Vishay Model P-350A Strain Indicator with a Terrametrics Switching Unit to enable switching from one axis reading to another (a backup unit to item a);

c. Orientation tool; placement tool, scribe, and placement rods (fifteen 12-foot sections, two 7-foot sections, two 3-foot sections, two 2-foot sections, two 1-foot sections, one master rod, and one leveling handle);

d. A male joint from an "A" rod attached to a 1-foot section of placement rod. This special adaptor was designed to be used if the standard placement rods proved unsatisfactory in placing and orienting the deformation gage. This special adaptor was designed to attach the placement rod, the "J" slot, and a set of oriented "A" rods. Together these three units would replace the standard placement rods for placing and orienting the deformation gage in the EX hole;

e. Calibration jig;

f. Gage accessories, including special pliers, O-rings, washers, two 250-foot unspliced cables, a 25-foot and 50-foot spare cable extensions, and a spare back case for standard USBM BDG;

g. Modified biaxial chamber, with an inner diameter of 5.25 inches and accessories including pressure dial and hand pump; and

h. Pajari and its accessories (for measuring the inclination of the hole from the vertical).

20. All of the equipment employed in this test program was calibrated to USBM standards as discussed by Hooker and Bickel (Ref. 2) and Fitzpatrick (Ref. 6).

Laboratory Test of the Standard USBM Borehole Deformation Gage

21. Prior to its use in the field, the USBM standard gage was tested to a water pressure of up to 100 psi (equivalent to a water depth of 230 feet) in a triaxial cell in Dames & Moore's Laboratory, Cranford, N. J. Three loading and unloading cycles were applied; the first in 10-psi increments, the second in 20-psi increments, and the third in 50-psi increments. A typical plot of pressure versus displacement and the schematic laboratory test setup are presented on Plate 8. The test results indicated that at 100 psi (~230 feet of water) the cantilevers were compressed by:

Axis I - 15,000 μ inches*

Axis II - 15,000 μ inches*

Axis III - 14,000 μ inches*

These cantilever deflections are well within the working range of the cantilever strain gages. However, the magnitude of this inward movement of the cantilevers may be significant in comparison to the possible linear range of the cantilever strain gages at depths greater than 200 feet. It should be noted that no correction factors are required as

---

* Field data indicated that at 154 feet of water head (66.7 psi), the cantilevers compressed: Axis I - 13,466 μ inches; Axis II - 14,038 μ inches; Axis III - 11,476 μ inches.

long as the readings corresponding to the deflected positions of the cantilevers are taken as the initial readings for the test. One of the major concerns of the laboratory test was to determine if the electrical components of the USBM standard gage would remain watertight at depths of up to 200 feet. The gage functioned properly during the laboratory tests under pressures corresponding to a water depth of 200 feet and during field tests up to a depth of 155 feet.

Calibration of the Borehole Deformation Gage

22. The three deformation gages were calibrated on a regular basis during the field testing program. A standard calibration jig was employed in calibrating the gages. The jig applied a known deflection to each cantilever, from which the calibration factor, K_i , was determined for each axis. The calibration record for each axis has been tabulated in Table 1 (K_i is given in units of 10^{-6} inches).

Calibration for the Biaxial Test

23. The biaxial chamber used in this program was calibrated using a specially constructed aluminum "core," 5.20 inches in diameter, and 16 inches in length (constructed using ASTM Standard Q.Q.A. 225-8 material). The aluminum "core" was manufactured by Terrametrics of Golden, Colorado. Testing of this core and all other rock overcores was performed using a Roylyn Pressure Gage with an accuracy of 0.25%. Over the full range of the gage (0 to 2000 psi), the test results were recorded with a precision of ± 5 psi. The average modulus for the aluminum "core" was 10.0×10^6 psi.

Computation of Stresses

24. To determine the stress from the recorded deformation readings, it is necessary to employ several equations based on the assumption of linear elasticity.

25. Multiplying the change in diameter as recorded by the strain indicator (indicator units R_1, R_2, and R_3) by the corresponding calibration factors (K_1, K_2, and K_3 in μ inches) gives the diametral change in three directions 60° apart. These values (U_1, U_2, U_3) with the modulus of elasticity, form the basis for evaluating the magnitude and direction of the maximum and minimum stresses (P_c and Q_c) acting in a plane perpendicular to the borehole axis. The maximum and minimum stresses are principal stresses only when the borehole is parallel to the third principal stress, which is not always true for vertical boreholes. However, under the given conditions of more or less homogeneous, gently dipping rocks of low relief, it can be assumed, with minimum error, that the third principal stress is vertical and equal to the overburden stress and that the minimum and maximum stresses perpendicular to the borehole are in fact the other two principal stresses.

26. The vertical stress can be calculated from:

$$\sigma_v = \gamma H \tag{1}$$

where

σ_v is the vertical stress (psf);

γ is the average density of the overlying material; and

H is the depth of measurement (ft).

To calculate the tectonic stresses in the rock mass, the gravitational stresses must be subtracted from the total measured stresses. For a vertical borehole, the magnitude of the lateral stress component of the overburden is given by the following expression:

$$\sigma_h = \sigma_v\left(\frac{\nu}{1 - \nu}\right) \tag{2}$$

where

σ_h is the horizontal stress;

σ_v is the vertical overburden stress; and

ν = Poisson's ratio (assumed equal to 0.25).

By substracting the estimated gravitational component of the stress

value from the calculated stresses the resulting tectonic and remanent stress field is determined. Equations based on a "Plane-Stress" analysis, as discussed by Obert and Duvall (Ref. 7), are used to determine the maximum (P_c) and minimum (Q_c) stresses normal to the borehole axis, as follows:

$$P_c = \frac{E}{6d}\left\{(U_1 + U_2 + U_3) + \frac{\sqrt{2}}{2}\left[\left(U_1 - U_2\right)^2 + \left(U_2 - U_3\right)^2 + \left(U_3 - U_1\right)^2\right]^{1/2}\right\} \quad (3)$$

$$Q_c = \frac{E}{6d}\left\{(U_1 + U_2 + U_3) - \frac{\sqrt{2}}{2}\left[\left(U_1 - U_2\right)^2 + \left(U_2 - U_3\right)^2 + \left(U_3 - U_1\right)^2\right]^{1/2}\right\} \quad (4)$$

where

- P_c is the maximum normal stress (psi);*
- E is the modulus of elasticity (psi);
- d is the "EX" hole diameter (inches); and
- U_1, U_2, U_3 are measurements of diametral deformation along three axes 60° apart. Deformation (inches) is positive for increasing diameter during overcoring;
- Q_c is the minimum normal stress (psi).*

27. The orientation of the principal stress axis is calculated from:

$$\theta_p = \frac{1}{2} \text{ arc tan } \frac{\sqrt{3}(U_2 - U_3)}{(2U_1 - U_2 - U_3)} \quad (5)$$

where

- θ_p is the angle from the U_1 axis (positive in a counterclockwise direction) to the major principal stress.

---

* Positive values of P_c and Q_c indicate compressive stresses.

The value of θ_p, together with the observed values of strain, permits the determination of the axes of maximum and minimum stresses.

28. The modulus of elasticity (E) from the biaxial testing is calculated from Fitzpatrick's (Ref. 6) "Plane-Stress" analysis, considering a "thick-wall" cylinder:

$$E_i = \frac{(4ab^2)(\Delta P_i)}{(b^2 - a^2)(\Delta U_i)} \tag{6}$$

where

- E_i is the modulus of elasticity (psi);
- i is the direction of the axis:
- a is the radius of the "EX" hole (inches);
- b is the radius of the core (inches);
- ΔP_i is the change in applied pressure (psi); and
- ΔU_i is the diametral deformation change during the pressure increment.

29. The procedure for the determination of the maximum and minimum stresses in Equations 3 and 4 assumes that the rock is isotropic with respect to the modulus of elasticity, that is, it has the same modulus value in all directions. This assumption is not always realistic; however, the biaxial test results (presented in Part IV) indicate that the assumption of isotropy is reasonable for these tests.

PART III: MODIFICATIONS

Modifications to Drilling and Testing Procedures

30. Following are the modifications made to the drilling procedures that are normally followed during overcoring operations. These modified procedures were adopted during this project:

- a. Instead of drilling the "EX" hole from the base of the 6-inch boring with the core barrel removed, the "EX" hole was drilled from inside the 6-inch core barrel. This task was accomplished by placing a stabilizer in the 6-inch core barrel, extending upward from the base of the the core barrel to the inside of the "NW" casing. The assembly consisted of a 6-foot "AX" flush joint casing with a 4-inch piece of hard plastic (a PVC "frisbee" with a central hole) at the base to keep it centered inside the 6-inch core barrel and a 2-inch thick piece of hard plastic (a PVC "frisbee" with a central hole) at the top to keep it centered inside the "NW" casing;
- b. Because of the moderately soft rock conditions at this site, the number of rotations per minute during overcoring was reduced from the recommended 120 rpm (Hooker and Bickel, Ref. 2) to approximately 50 rpm, after experiencing difficulties in the initial tests. This reduction in the number of rotations was intended to reduce the chatter of the drill rods and thus reduce the changes of core breakage during overcoring. A penetration of 1/2 inch per 40 to 60 seconds was still maintained;
- c. At the end of the test, the rotation of the overcoring drill bit was stopped (however, the water was allowed to flow in order to maintain a nearly uniform thermal environment) while recording any time-dependent stress relief behavior of the rock core. This procedure was implemented so as not to cause breakage of the core after a test was complete, yet permit continuous monitoring of the core deformation; and
- d. Whenever possible, the scribing tool was used to feel the presence of fractures in the "EX" hole. The information interpreted from this "feel technique" was compared with the "EX" core for possible correlation. Most of the time, it was possible to detect the presence and location of fractures by this procedure, which otherwise would have been difficult by examination of the "EX" core only.

Modification to the USBM Borehole Deformation Gage

31. One major problem that may be encountered at depths greater than 200 feet is that the cantilevers of the BDG will be compressed as a result of the hydrostatic pressure inside the borehole. As mentioned in paragraph 21, the magnitude of this inward movement of the cantilevers may be significant in comparison to the possible linear range of the cantilever strain gages.

32. To circumvent the above problem, it was decided to equalize the water pressure both inside and outside the BDG chamber by drilling holes through the gage wall and letting the water enter the chamber. This modification, while equalizing the pressure on all sides of the cantilever, gives rise to another problem; that is, it exposes the wiring on the cantilevers to water, which may result in shorting of the electrical circuits if proper waterproofing is not implemented.

33. In addition to the routine measures taken to cover the wirings with waterproofing sealants, specially molded neoprene rubber membranes were used to cover the cantilevers as well as the wiring (the neoprene rubber membrane and its applicability was first discussed by Austin (Ref. 4).

34. Certain mechanical modifications to the BDG were also implemented. The total length of the BDG was increased to provide greater stability of the gage inside the "EX" hole (the increase in length was divided between the extended 'nose' or the front piece and the back piece). Instead of the usual three springs on only the bottom of the gage, six springs each were provided on both top and bottom of the gage. These springs were intended to provide vertical positioning of the gage in the "EX" hole, while overcoring was in progress. The details of the modified borehole deformation gage are shown on Plate 9 (for the sake of comparison a standard USBM BDG and a modified Terrametrics BDG are also shown on Plate 9).

Performance of Borehole Deformation Gages

35. Three borehole deformation gages were used during the field

testing program: a USBM standard borehole deformation gage, a modified USBM borehole deformation gage, and the Terrametrics borehole deformation gage (all manufactured by Terrametrics, Golden, Colorado).

36. Of the three gages used, the USBM standard borehole deformation gage was the only one which performed reasonably well at all test depths. The only precaution required was to make sure that the O-rings on the buttons were properly lubricated and that no piece of the outer O-ring was damaged by the sharp edges of pliers. Whenever the O-ring was damaged, water seepage into the BDG chamber invariably occurred.

37. Usage of the modified gage proved to be somewhat unsuccessful due to the following reasons:

- a. While testing the modified BDG in the field at the ground surface, it was noticed that when pressure was applied to one set of cantilevers, movement in the other two cantilevers resulted, because all the cantilevers were circumscribed by the rubber boot and hence were not totally independent of one another;
- b. The compressed air entrapped inside the rubber boot prevented proper calibration of the gage and caused difficulties in establishing initial readings. To force the air out of the boot, the gage was placed in the "EX" hole for 10 to 15 minutes. The gage was then retrieved, calibrated and reinserted in the hole at the test depth, and the initial reading was taken. However, there was no way of ensuring the complete purging of the entrapped air; and
- c. Leakage occurred in several of the rubber membranes which were specially molded for this project. Each membrane was pressure tested with a water hose. Approximately 60 percent of the boots ruptured under pressure and leaked. Even after finding a good rubber boot which could withstand the high water pressure, the cantilever system shorted out (the electronic short circuit of the strain gages on the cantilevers may have been caused by the waterproofing material used*).

38. The Terrametrics deformation gage was placed in the "EX" hole twice. After the second time, the gage shorted out. The problem

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* Before delivery of the modified gage to Dames & Moore, Terrametrics encountered difficulty trying to waterproof the modified gage and had to rebuild the strain gage-cantilever system several times.

stemmed from a shorting in the female part of the cable couple in the BDG. A proper evaluation of the Terrametrics BDG could not be made, because no test could be run to completion using the Terrametrics gage.*

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* Dames & Moore has previous experience with the Terrametrics BDG, and it is felt that some of the design features of this gage have successfully circumvented some of the limitations of the standard USBM BDG. However, more refinement is needed.

PART IV: RESULTS AND DISCUSSION

Field Tests

39. A total of 15 tests were attempted, with four tests showing successful results (Table 2). Eight tests could not be completed because of core breakage during overcoring operations (Tests 1, 3, 5, 6, 8, 9, 10, and 13). One other test was unsuccessful due to the gage slipping in the "EX" borehole (Test 7). Two other tests were unsuccessful due to instrumentation malfunctions (Tests 11, 12).

40. The close jointing and fracturing in the shale-sandstone was responsible for the high percentage of unsuccessful tests. The total number of incomplete tests caused by core breakage was minimized by detecting possible fractures in the "EX" borehole while scribing the "EX" hole (this procedure was mentioned as a footnote in Part II).

41. The in situ tests which are considered meaningful are Tests 2, 4, 14, and 15 (Tables 3 and 4 and Plates 10 to 13). Tests 2 and 4 were performed in red shale and red siltstone, respectively. These two tests were conducted above a *highly fractured gray* shale zone located between the depths of 104 feet and 111 feet. The measured stresses above this zone are relatively low; the major principal stresses are +396 psi and +339 psi, respectively, while the minor principal stresses are +15 psi and +165 psi, respectively. The orientation of the major principal stress is N46°E and N37°E, respectively (Table 3).

42. Tests 14 and 15 were conducted below the fractured zone (104 to 111 feet). Test 14 was performed at 148' 3-1/2" (corresponding to mid-tunnel depth) and Test 15 was conducted at 154' 11-1/2" (bottom of tunnel). The magnitudes of calculated maximum stresses increased to +488 psi and +585 psi, respectively, for these two tests. The calculated minor principal stresses were -42 psi and +133 psi, respectively. The orientations of the major principal stresses were N60°E and N48°E, respectively (Table 3).

43. The average indicates that the maximum principal stress (P_c) is oriented N48°E and has a magnitude of +452 psi (Table 3).

44. Two Pajari tests for verticality were performed in the 6-inch borehole at 78' 10" and 155' 2". At 78' 10" the 6-inch borehole was 1.4 feet off the vertical with an azimuth of N32°E, and at 155' 2" the borehole was 1.3 feet off with an azimuth of N16°E.

Modulus Tests

45. Modulus of elasticity tests using a biaxial cell were attempted on rock cores from the following locations:

- a. At a depth of 83' 3-1/2" to 84' 2-1/2" (core obtained from below Test 2);
- b. From Test 4;
- c. From Test 14; and
- d. From Test 15.

The determination of the modulus was completely successful only for the rock core from Test 15 (gray shale). Other tests performed on the red shale, siltstone, or shale-sandstone were not totally successful because the cores fractured during the initial stages of loading or during the second cycle of loading (Table 5).

46. To obtain the modulus of the rock for which the biaxial test fails, the normal practice is to perform a uniaxial compression test on "NX" core from nearby exploratory boreholes. Unfortunately, there were no "NX" boreholes drilled in the immediate near area; therefore, the modulus of elasticity was estimated from the data compiled on the 5-inch core, prior to failure.

47. The only 5-inch core that exhibited slightly anisotropic characteristics was obtained from Test 14; all other cores tested were found to be isotropic. For the purposes of analysis in this project, all the cores were assumed to be isotropic. The results of the biaxial testing program are shown in Plates 14 through 22.

Regional Correlation

General

48. Several components contribute to the total in situ stress

field measured at any one location. The most important are: (a) a gravitational component derived from the mass of overburden; (b) residual and/or remanent component(s), which may or may not be interrelated, derived from conserved elastic strain energy locked into the rock; and (c) a regional or current tectonic component derived from applied stress.

49. The gravitational component in a horizontal plane can be calculated from the depth of measurement and Poisson's ratio. For an average rock density of 140 pcf for shale/siltstone and a Poisson ratio of 0.25, the stress component in the horizontal direction would be equal to 39 psi at 120 feet, an average depth for the test data recorded during this investigation.

50. At present, however, there is no quantitative method to separate the effects of the remanent component from the current tectonic component.

Orientation of maximum horizontal compression in northeastern North America

51. The average azimuth of the horizontal component of maximum compressive stress reported in the geologic literature for northeastern North America is plotted on Plate 23. These orientations were obtained by the strain relief and hydrofracture techniques of measuring in situ stress. The magnitude and orientation of the principal stresses presented in the literature are summarized in Tables 6, 7, and 8.

52. For the northeastern United States, Sbar and Sykes (Ref. 8, 9, and 10) concluded that: (a) the maximum compressive stress trends east to northeast over an area extending from west of the Appalachian Mountain system to the middle of the continent, and from southern Illinois to southern Ontario; and (b) the stress pattern is different and not as simple in the Appalachian Mountain system as in the adjacent region to the west.

53. Measurements A, B, Ca, and V (Plate 23), west of the Appalachian fold and thrust belt, support the presence of a uniform regional component of compression that trends about east-west in the upper part of the lithosphere.

54. The direction of maximum compression (Pc) in the vicinity of Lake Ontario, however, appears to be variable. Measurements VI and 11 recorded a northeast trend, while VII and 10 indicate a northwest trend (Plate 23).

55. The stress pattern appears to be complicated in the Appalachian Mountain system. The suggestion of Hooker and Johnson (Ref. 11) that the direction of the major principal stress may be aligned with the structural trend of the Appalachians is not entirely supported by the orientations of Pc plotted in Plate 23. About 50% of the measurements, #1, #2, #4, #5, #6, #7, and #13, recorded a northerly-trending Pc, while the remaining 50%, #3, #9, #14, #15, #16, #17, and #18, recorded a northeasterly-trending Pc (Plate 23). These measurements appear to record the resultant of more than one stress component (residual, gravitational, remanant, and contemporary), such that a locally predominant stress component would influence the trend of Pc at any one site.

56. Engelder and Sbar (Ref. 12) suggested that the strain relieved following initial in situ overcore at Barre, Vermont, contains large components of residual strain. If different outcrops have scattered orientations of residual strain, initial overcore would probably yield scattered orientations of Pc. Engelder and Sbar (Ref. 12) have attempted to discriminate between applied and residual strains, using the double overcoring technique, in northern New York State and at Barre, Vermont (Plate 23). Presumably the initial overcore relieves both applied and part of the residual strains, whereas the second overcore relieves just residual strains. Residual strains obtained from the second overcore may enable the separation of residual effects from the combination of applied and residual strains relieved during the first overcore.

57. The relief of both macroscopic and microscopic residual strain following the initial overcore must be larger than the relief of just microscopic strain by the second overcore. Therefore, the magnitude of residual strain obtained with the double overcore was assumed to be between 1/3 and 1 of the residual strain relieved by the initial

overcore (Engelder and Sbar, Ref. 12). The correction to the initial field measurement was applied by subtracting between 1 and 4 times each of three components of strain, following double overcore, from their respective components of strain measured upon initial overcoring. The results of the 4 times double overcore strain subtraction are listed in Table 8. To date, there is no model which adequately treats the relief of superimposed residual strains in order to isolate the applied component related to the regional stress pattern.

58. The magnitude and orientation of the principal stresses measured during this investigation are considered compatible with the data reported in the literature. The trend of Pc at the site is parallel to most of the trends measured in southeastern New Hampshire, northeastern Massachusetts, and southeastern New York (Plate 23). The magnitude of Pc, however, is smaller than the magnitudes measured in southeastern New Hampshire and northeastern Massachusetts (Table 6).

59. The magnitude of the average maximum horizontal stress at depth in North America has been represented by:

$$\sigma_H = (620 \pm 116) \text{ psi} + (1.690 \pm 0.311) \text{ psi/ft depth} \tag{7}$$

(Lindner and Halpern, Ref. 13)

$$\text{and} \quad \sigma_H = 580 \text{ psi} + 0.95 \text{ psi/ft depth} \tag{8}$$

(Haimson, Ref. 14)

60. If the above two approaches are used for the average depth of measurement at the site (120 feet), the average value of the major principal stress in the horizontal direction is expected to be 825 psi and 694 psi, respectively. The average value calculated from actual measurements at the site is 452 ± 133 psi (Table 6).

PART V: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

61. Deep borehole in situ stress tests, at depths greater than 100 feet, were performed at a site in the City of Hartford, Connecticut. The main purpose of the investigation was to demonstrate the feasibility of performing in situ stress tests at depths greater than 100 feet using overcoring techniques. In order to accomplish this purpose, modifications were made to the borehole deformation gage, equipment used for performing the field test, and the testing and drilling procedures. Tests were successfully performed at a depth of 155 feet. Because of the highly jointed and fractured nature of the rocks at the site, a number of tests could not be carried to completion. There were some instances of instrumentation malfunction. However, at the end of the project it was felt that with minimal modification to the borehole deformation gage and the overcoring procedures, in situ stress tests could be performed at depths greater than 150 feet. Based on our experience to date, if reasonably good rock conditions exist, in situ stress tests using overcoring procedures can be performed up to a depth of 200 feet with minimal difficulty.

Conclusions

62. Equipment and procedures:

<u>a</u>. The standard borehole deformation method, which uses overcoring techniques, can be employed to measure in situ stresses up to depths of 150 to 200 feet, with little or no modification to the design of gage and procedures;

<u>b</u>. Extended nose and provision of extra springs at top and bottom of a borehole deformation gage are useful modifications to the gage, which result in greater stability, and hence better performance;

<u>c</u>. Exposure of the cantilevers of the borehole deformation gage to water by providing holes through the wall of the gage does prevent the compression of cantilevers caused by high water pressures at great depths. However, this

creates serious problems of waterproofing the strain gages and other electronic circuits;

d. The use of neoprene rubber membranes (boots) to cover the cantilevers does not serve any useful purpose of rendering the cantilevers waterproof;

e. Placement and retrieval of the borehole deformation gage as well as its orientation in the borehole, and scribing of the "EX" hole, can all be done manually using standard procedures up to a depth of 200 feet; and

f. Several alternate procedures and backup equipment are necessary to perform deep borehole in situ stress tests.

63. Based upon the results of four successful in situ stress tests between the depths of 80 and 155 feet at the test site in the City of Hartford, Connecticut, we conclude that:

a. There appears to be relatively low lateral stresses at the site of investigation (average maximum principal stress of 452 $\pm$ 133 psi, compression);

b. The stresses of lower magnitudes occur above the fractured zone (the fractured zone is between 104 and 111 feet at the test hole location), and the stresses of slightly higher magnitudes occur below the fractured zone;

c. The magnitudes of stresses increase with depth, which seems to be consistent with known past measurements; and

d. The trend of the maximum principal stress appears to be consistent with the existing regional data (average orientation is in the direction N48°E $\pm$ 12°).

Recommendations

64. On the basis of the experience gained during this project, the following recommendations are made:

a. An "NX" borehole should be drilled near the location of the test borehole. The "NX" borehole is invaluable in determining the depth at which an in situ stress test should be attempted and hence will save valuable field time;

b. Stabilizers should be placed on the drill rods so that whipping motion of the rods inside the "NW" casing is reduced, which also assures the concentricity of the "EX" hole with the 6-inch hole;

c. Since the technique of overcoring requires as nearly a vertical hole as possible, new drill rods should be used. New drill rods should eliminate whipping and reduce the possibility of the 6-inch and "EX" boreholes shifting away from the vertical;

d. During the project, there were instances when small rock fragments wedged the gage in the "EX" borehole, making it impossible to attach the 'J' slot retrieval tool to the rear of the gage. The total length from the bottom of the gage to the studs at the rear of the gage (where the 'J' slot on the retrieving rod attaches to the gage) is 7-3/4 inches, leaving only 1-1/2 inches of the BDG extending above the base of the 6-inch hole (this is true only if the 5-inch core breaks flush at the bottom and if the gage is placed 8 inches below the bottom of the 6-inch borehole). Usually, due to the irregular breakage of the 5-inch core, little or no portion of the gage protrudes above the bottom of the 6-inch hole. Therefore, it is recommended that the rear section of the gage be made longer (approximately 6 to 8 inches longer) so that if small rock fragments fall against the gage, the gage will not be wedged beyond retrieval; and

e. More research is required to develop an inexpensive and expendable borehole deformation gage which will perform under the severe test conditions. Development of such a gage is very important, because, in a given deep borehole in situ stress test, the bulk of the money is spent on drilling costs and the time spent by the field crew. Any attempt directed towards saving on this item is well worth the cost of several gages.

REFERENCES

1. Department of the Army, New England Division, CE, "Park River Local Protection, Connecticut River Basin, Hartford, Connecticut," Design Memorandum No. 9, Sep 1976, Waltham, Mass.

2. Hooker, V. E. and Bickel, D. L., "Overcoring Equipment and Techniques Used in Rock Stress Determination," IC 8618, 1974, U. S. Bureau of Mines, Washington, D. C.

3. American Society for Testing and Materials, "Determination of Stress in Rock--A State of the Art Report," Special Technical Publication No. 429, pp 28-33, 1966.

4. Austin, W. G., "Development of a Stress Relief Method with a Three-Directional Borehole Deformation Gage," REC-OCE-70-10, Mar 1970, U. S. Bureau of Reclamation, Denver, Colo.

5. Terrametrics, "Instruction Manual, Borehole Deformation Gage, Terrametrics Type," 1977, Golden, Colo.

6. Fitzpatrick, J., "Biaxial Device for Determining the Modulus of Elasticity of Stress-Relief Cores," R.I. 6128, 1962, U. S. Bureau of Mines, Washington, D. C.

7. Obert, L. and Duvall, W. I., Rock Mechanics and the Design of Structures in Rock, Wiley, New York, 1967.

8. Sbar, M. and Sykes, L. R., "Contemporary Compressive Stress and Seismicity in Eastern North America: An Example of Intraplate Tectonics," Geological Society of America Bulletin, Vol 81, 1973, pp 1861-1882.

9. Sykes, L. R. and Sbar, M. L., "Intraplate Earthquakes, Lithospheric Stresses and the Driving Mechanism of Plate Tectonics," Nature 245, No. 5424, Oct 12, 1973, pp 298-302.

10. Sykes, L. R. and Sbar, M. L., "Focal Mechanism Solutions of Intraplate Earthquakes and Stresses in the Lithosphere," Geodynamics of Iceland and the North Atlantic Area, Reidel Publishing Co., Holland, 1974, pp 207-224.

11. Hooker, V. E. and Johnson, C. F., "Near-Surface Horizontal Stresses Including the Effects of Rock Anisotropy," R.I. 7224, 1969, U. S. Bureau of Mines, Washington, D. C.

12. Engelder, J. T. and Sbar, M. L., "Determination of the Regional Stress Patterns in New York State and Adjacent Areas by In Situ Strain Relief Methods," Annual Technical Report Prepared by ERDA, 1976.

13. Lindner, E. N. and Halpern, J. A., "In Situ Stress: An Analysis," Proceedings, 18th U. S. Symposium on Rock Mechanics, Keystone, Colorado, Vol 2, 1977.

14. Haimson, B. C., "The Hydrofracturing Stress Measuring Technique: Method and Recent Field Results in the U. S.," Symposium, International Society for Rock Mechanics, Australia, 1976.

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Table 1

Calibration of Borehole Deformation Gages

Gage No.	Cable	Date	Gage Factor	K_1 $x10^{-6}$ in.	K_2 $x10^{-6}$ in.	K_3 $x10^{-6}$ in.
27*	#1-250'	5-24-77	0.40	1.09	1.06	1.12
27*	#1-250'	6-03-77	0.40	1.11	1.11	1.08
27*	#1-250'	6-15-77	0.40	1.13	1.12	1.07
27*	#1-250'	6-20-77	0.40	1.13	1.10	1.06
30**	#2-250'	6-22-77	0.80	3.20	3.37	3.09
22***	#3-250'	6-24-77	0.40	4.72	4.23	4.19
22***	#3-250'	6-24-77	0.20	2.41	2.10	2.05
32*	#2-250'	6-27-77	0.40	1.09	1.05	1.07
32*	#2-250'	6-30-77	0.40	1.08	1.05	1.08
32*	150'	7-08-77	0.40	1.03	1.01	1.02

* Standard USBM Borehole Deformation Gage.
** Modified USBM Borehole Deformation Gage.
*** Terrametrics Borehole Deformation Gage.

Table 2

Summary of Tests Attempted

Test No.	Depth	Date	Inches Overcored	Comments
1	80' 8"	5-31-77	1½ inches	Core broke at 1½ inches of overcoring
2	80' 8½"	5-31-77	13 inches	Successful test; core broke at 80' when taking out from core barrel
3	82' 9½"	6-01-77	12 inches	Core broke at 82'9½"
4	92' 7½"	6-03-77	13 inches	Successful test
5	100' 6"	6-08-77	3½ inches	Core broke at 3½ inches of overcoring
6	125' 0"	6-14-77	11-3/4 inches	Core broke at 124'10½"
7	132' 6"	6-16-77	1½ inches	Gage moved in EX hole
8	132' 7"	6-17-77	0 inches	Core "stickup" broke
9	132' 9"	6-17-77	11½ inches	Core broke at 132'9"
10	136' 0"	6-20-77	0 inches	Core "stickup" broke
11	147' 7½"	6-23-77	0 inches	Water entered rubber"boot" (modified gage)
12	147' 7½"	6-24-77	0 inches	Gage shorted out (Terrametrics gage)
13	147' 7½"	6-24-77	2 inches	Core broke at 2 inches of overcoring
14	148' 3½"	6-30-77	14 inches	Successful test
15	154' 11½"	6-30-77	13 inches	Successful test (gage may have moved due to drill rig stalling)

Table 3

Test Results

Test No.	Depth	Diam. Deformation ($x10^{-6}$ in.) U_1	U_2	U_3	Average Elastic Modulus $x10^6$ psi	P_c psi	Q_c psi	Bearing of P_c	Rock Type
2	80' 8½"	+556	- 68	+115	3.07*	+396	+ 15	N46°E	Red Shale/Mudstone
4	92' 7½"	+416	+165	+158	3.07	+339	+165	N37°E	Red Siltstone
14	148' 3½"	+218	-104	+129	8.25	+488	- 42	N60°E	Red Shale/ Sandstone
15	154' 11½"	+321	+ 7	+113	7.33	+585	+133	N48°E	Black Shale
			Average Values		6.22**	+452	+68	N48°E	

* Modulus value from Test 4.

** Average of three tests.

Table 4

Deformation Calculations

Test No.	$U_{1\theta}$	R_1	R_2	R_3	K_1 $\times 10^{-6}$ in.	K_2 $\times 10^{-6}$ in.	K_3 $\times 10^{-6}$ in.	U_1 $\times 10^{-6}$ in.	U_2 $\times 10^{-6}$ in.	U_3 $\times 10^{-6}$ in.
2	N38°E	+510	- 64	+103	1.09	1.06	1.12	+556	- 68	+115
4	N38°E	+375	+149	+146	1.11	1.11	1.08	+416	+165	+158
14	N38°E	+200	- 99	+121	1.09	1.05	1.07	+218	-104	+129
15	N38°E	+297	+ 7	+105	1.08	1.05	1.08	+321	+ 7	+113

Notes: 1) $U_{1\theta}$ *is the orientation of axis I with respect to north;*

2) R_1, R_2 and R_3 are indicator readings corresponding to axes I, II, and III;

3) K_1, K_2 and K_3 are the calibration factors corresponding to axes I, II, and III; and

4) U_1, U_2 and U_3 are the deformations in microinches along the three axes 60° apart.

Table 5

Biaxial Test Results

In Situ Test No.	Depth From - To	Axis 1 E_1 $x10^6$ psi	Axis 2 E_2 $x10^6$ psi	Axis 3 E_3 $x10^6$ psi	$U_{1\theta}$	Comments
Below 2	83' 3½" - 84' 2½"	--	--	--	N52°W	Core fractured at 600 psi
4	91' 9" - 92' 8½"	2.92	3.20	3.08	N52°W	
14	147' 4½" - 148' 10½"	8.88	8.87	6.99	N38°E	Core fractured at 120 psi
15	154' 5" - 155' 6"	7.51	7.32	7.17	N38°E	

Table 6

Some In Situ Secondary Principal Stresses in a Horizontal Plane, Eastern North America

(Strain Relief Measurements Using U. S. Bureau of Mines Strain Gage)

No.	Location	Reference*	Depth to Collar of Hole, ft**	Depth of Measurement ft	P_c psi	Q_c psi	Trend Pc	Rock Type
1a	Barre, Vt.	5	0	0.5-1.5	510	107	N32E	Granite
1b	44°11'N 72°32'W		300	0.8-2.0	2958	1353	N08W	Granite
	Calculated Average	5		150	1734	791	N14E	Granite
2	Proctor, Vt.	5	0	0.5-1.8	1584	414	N09E	Dolomite
	43°40'N 73°07'W		0	0.5-1.8	1072	617	N22W	Dolomite
	Calculated Average	5		1.2	1328	516	N04W	Dolomite
3a	Chelmsford, Mass.	5	250	0.5-1.5	4530	2343	N53E	Granite
3b			30	0.5-1.8	1654	486	N43E	Granite
3c	42°35'N 71°21'W		0	6.0	413	279	N30W	Granite
3d			4	3.0	1392	585	N40E	Granite
3d			4	3.5	1503	596	N42E	Granite
3e			50	2.0	1181	872	N73W	Granite
3e			50	2.5-3.7	1133	781	N78W	Granite
3e			50	4.2	1818	1010	N85W	Granite
3f			70	2.0	3215	1845	N60E	Granite
3f			70	2.5-3.7	3275	1515	N63E	Granite
3f			70	4.2	3346	1927	N56E	Granite
	Calculated Average	5		61.9	2133	1113	N56E	Granite

(Continued)

* Numbered references are listed on last page of this table.

** When drilling was initiated at varying depths in some quarries, the vertical distance from the original rock mass surface is indicated.

(Sheet 1 of 5)

Table 6 (Continued)

No.	Location	Reference	Depth to Collar of Hole, ft	Depth of Measurement ft	P_c psi	Q_c psi	Trend Pc	Rock Type
4a	Tewksbury, Mass.	5	0	4.0-5.0	444	150	N05E	Paragneiss
4b	42°40'N 71°13'W		40	4.7	1339	978	N22E	Paragneiss
4b			40	5.6	1056	678	N20W	Paragneiss
4b			40	7.5	1313	611	N06E	Paragneiss
4b			40	8.0	1792	897	N11W	Paragneiss
	Calculated Average	5		38.1	1189	663	N02W	Paragneiss
5	Nyack, N. Y. 41°04'N 73°55'W	6	--	0.5-1.5	173	68	N02E	Diabase
6	St. Peters, Pa.	5	0	4.3	882	318	N07E	French Creek
	40°13'N 75°41'W		0	4.8	820	335	N14E	Norite
7a	Rapidan, Va.	5	26	0.5-1.7	1380	905	N02W	Diabase
7a	38°22'N 78°03'W		26	0.5-1.5	1236	647	N10E	Diabase
7b			36	2.0	1933	1754	N09W	Diabase
7b			36	2.5	1909	1661	N14E	Diabase
7b			36	3.5-4.3	1838	1704	N54E	Diabase
7b			36	4.9	1770	1637	N18E	Diabase
	Calculated Average	5		8.6	1678	1385	N06E	Diabase
8a	Mt. Airy, N. C.	5	50	1.0-3.0	3990	950	N86E	Granite
8b			30	0.5-1.5	2352	549	N78W	Xenolith (5 x 2 ft)
8b	36°27'N 80°35'W		30	1.7-2.5	2205	1152	N86W	Granite
8b			30	0.5-1.5	1635	874	N89W	Granite
8b			30	0.5-1.5	2594	1823	N89W	Granite
8c			20	0.5-1.5	2005	1799	N46W	Granite

(Continued)

(Sheet 2 of 5)

Table 6 (Continued)

No.	Location	Reference	Depth to Collar of Hole, ft	Depth of Measurement ft	P_c psi	Q_c psi	Trend Pc	Rock Type
	Calculated Average	5		33	2464	1191	N87E	Granite
9a	Seabrook, N. H.	4		33.8	1335	1025	N38E	Pegmatite Vein
9b				36.8	150	50	N55W	Quartz Diorite
9b				38.3	1190	850	N03E	Quartz Diorite
9b				39.3	2150	1570	N45E	
9b				41.4	1400	800	N48E	Quartz Diorite
	Calculated Average	1					N52E	
10a	Somerset, N. Y.	2		27.7	640	460	N15W	Sandstone
10a				32.6	295	60	N60W	Sandstone
10a				71.3	340	170	N15W	Sandstone
10b				29.4	450	240	N15W	Sandstone
10b				66.1	265	-40††	N10W	Sandstone
10b				67.3	500	140	N15E	Sandstone
	Calculated Average	1					N47W	
11	Niagara Falls, N. Y.	7†		<150	993	-10††	N55E	Dolomite
12	Elliot Lake, Ont.	3	934-1312		2987	2560	EW	Sandstone
			984		5263	2845	NE	Sandstone
	$46^{o}28'N\ 82^{o}00'W$		2297		5263	3271	EW	Sandstone

(Continued)

† Conversion factors used: 1 bar = 14.6 psi and 1 metre = 3 ft.
†† Negative sign indicates tensile stress.

Table 6 (Continued)

No.	Location	Reference	Depth to Collar of Hole, ft	Depth of Measurement ft	P_c psi	Q_c psi	Trend Pc	Rock Type
13	Morgantown, Pa.	7†	2300		7446	584	N27E	Diabase
14	Lake Tiorati, N. Y. Calculated Average	1		20	221 ± 7	-25 ± 127	N44E ± 18	Precambrian Gneiss
15	Cedar Flats, N. Y. Calculated Average	1		39.1	201 ± 188	94 ± 191	N68E ± 27	Hornblende Gneiss
16	Briarcliff Manor, N. Y. Calculated Average	1		38.8	1122 ± 355	33 ± 267	N66E $\pm 11^{o}$	Paleozoic Manhattan Schist
17	Philipstown, N. Y.	1		51.6	315 ± 216	20 ± 246	N28E ± 14	Precambrian Gneiss
18	Sterling, N. Y.	8		33.2-59.31	1350 ± 150	800 ± 150	N48E ± 25	Sandstone
19	Montague, Mass.	9		35.7-68.6	1170 ± 800	800 ± 700	N15W ± 40	Sandstone
20	Northfield Mt., Mass.	10	580-661	5-8	4380 ± 3000	1650 ± 700	N70E ± 17	Gneiss
21	Bear Swamp, Mass.	11	1250	5-30	1700	750	N84W	Gneiss/ shist
22	Hartford, Connecticut $41^{o}45.5'N$ $72^{o}40'W$ Calculated Average	This report		119.2	452 ± 133	68 ± 110	N48E ± 12	Shale/ siltstone

(Continued)

† Conversion factors used: 1 bar = 14.6 psi and 1 metre = 3 ft.

Table 6 (Concluded)

No.	Location	Reference	Depth to Collar of Hole, ft	Depth of Measurement ft	P_c psi	Q_c psi	Trend Pc	Rock Type

References

1. Dames & Moore (1977), "Geotechnical Investigation of the Ramapo Fault System in the Region of the Indian Point Generating Station."

2. Dames & Moore (1974), "Preliminary Safety Analysis Report of the Somerset Nuclear Power Station," for New York State Electric and Gas Company.

3. Eisbacher, G. H. and H. U. Bielenstein (1971), "Elastic Strain Recovery in Proterozoic Rocks near Elliot Lake, Ontario," J. Geophys. Res., Vol. 76, pp 2012-2021.

4. Geotechnical Engineers, Inc. (1973), "Rock Stress Measurements in Boring OC1A, Seabrook Station," for Yankee Atomic Electric Company and Public Service Company of New Hampshire.

5. Hooker, V. E. and C. F. Johnson (1969), "Near-Surface Horizontal Stresses Including the Effects of Rock Anisotropy," U. S. Bureau of Mines Report of Investigation, R.I. 7224, 29 pp.

6. Hooker, V. E. and C. F. Johnson (1967), "In-Situ Stresses Along the Appalachian Piedmont," 4th Canadian Symposium on Rock Mechanics, Ottawa.

7. Sbar, M. L. and Lynn R. Sykes (1973), "Contemporary Compressive Stress and Seismicity in Eastern North America, An Example of Intra-Plate Tectonics," Geol. Soc. Am. Bull., Vol 84, pp 1861-1882.

8. Dames & Moore Reports on Parr (1974), South Carolina; on Sterling, New York; on Limerick, Pennsylvania; on Mineral, Virginia; and on Somerset, New York.

9. Northeast Nuclear Energy Co. (1974), Montague Nuclear Power Station, Units 1 and 2, PSAR, Docket 50496-4, 272 p.

10. Lucius Pitkin, Inc. (1969), In-Situ Stress Measurements, Northfield Mountain Pumped Storage Project, Franklin Co., Mass., Technical Report 6048.

11. Foundation Sciences, Inc. (1971), Bear Swamp Project, Rock Mechanics Studies, for New England Power Service Co., p 144.

(Sheet 5 of 5)

Table 7

Some In Situ Secondary Principal Stresses in a Horizontal Plane, Eastern North America

(Hydraulic Fracturing Measurements)

No.	Location	Reference*	Depth of Measurement ft	Q_c psi	Trend of Pc	Rock Type
A	60 miles south of Pittsburgh, Pa.	1	Approximately 700	800 to 1,000	096 ± 5	Sandstone and mudstone
B	Bradford, Pa.	3	-	-	Average (11)** N70E	Sandstone
Ca	Alma Township N. Y.	2	Average (3) 1655	2013	Average (3) N77E	Sandstone
Cb	Falls Township Hocking County	2	Average (4) 2653	2217	Average (4) N64E	Sandstone

* Numbered references are listed below:

1. Dahl, H. D. and R. C. Parsons (1972), "Ground Control Studies in the Humphrey No. 7 Mine," Christopher Coal Div., Consolidated Coal Co.; Transactions Society of Mining Engineers, AIME, Vol 252, p 211-222.
2. Haimson, B. and E. J. Stahl (1969), "Hydraulic Fracturing and the Extraction of Minerals Through Wells; in the Third Symposium on Salt," Northern Ohio Geol. Soc. p 421-432.
3. Overbey, W. K., Jr., and R. L. Rough (1968), "Surface-Joint Patterns Predict Well-Bore Fracture Orientation," Oil and Gas Journal, Vol 66, p 84-86.

** (11) Number of measurements.

Table 8

Some In Situ Strain Relief Measurements Recorded at the Surface, Northeastern North America (Strain Relief Measurements Using Foil Resistance Strain Gage Rosettes Bonded to Rock)

No.	Location	Reference*	Average Orientation of Maximum Expansion (Trend of Pc)	Rock Type
I	Measurements at 4 sites between Plattsburgh, N. Y. and Canadian Border	1	N78W	Keeseville member Potsdam sandstone
II	Approximately 10 miles SSW of Plattsburgh, N. Y.	1	N08E	Nicholville member Potsdam sandstone
III	Barre, Vt.	2	** Initial: N55W † Computed: N04W	Granite
IV	Plattsburgh, N. Y.	2	Initial: N76E Computed: N64W	Potsdam sandstone
V	St. Johnsville, N. Y.	2	Initial: N61E Computed: N87W	Precambrian Gneiss and Paleozoic limestone
VI	Alexandria Bay, N. Y.	2	Initial: N44E Computed: N65E	Lower Paleozoic sediments Potsdam sandstone
VII	Brockport, N. Y.	2	Initial: N10W Computed: N60W	Paleozoic sediment sandstone, shale and limestone

(Continued)

* References made here are listed at the end of this table.

** Initial: Average of initial field measurements.

† Computed: Average estimate of applied strain after subtracting 4 times each of three components of strain, following double overcore, from their respective components of strain measured upon initial overcoring.

Table 8 (Concluded)

No.	Location	Reference	Average Orientation of Maximum Expansion (Trend of Pc)	Rock Type

References

1. Engelder, J. T. and M. L. Sbar (1976), "Evidence for Uniform Strain Orientation in the Potsdam Sandstone, Northern New York, From In-Situ Measurements," Journal of Geophys. Res., Vol 81, pp 3013-3017.

2. Engelder, J. T. and M. L. Sbar (1976), "Determination of the Regional Stress Patterns in New York State and Adjacent Areas by In-Situ Strain Relief Measurements," Annual Technical Report prepared for New York State Energy Research and Development Authority, 124 p.

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL
DEPTH CURVES AND SOUNDINGS IN FEET—DATUM IS MEAN LOW WATER
SHORELINE SHOWN REPRESENTS THE APPROXIMATE LINE OF MEAN HIGH WATER
THE MEAN RANGE OF TIDE IS APPROXIMATELY 1.7 FEET

SITE LOCATION PLAN

REFERENCE: (HARTFORD NORTH QUADRANGLE,
USGS SERIES)

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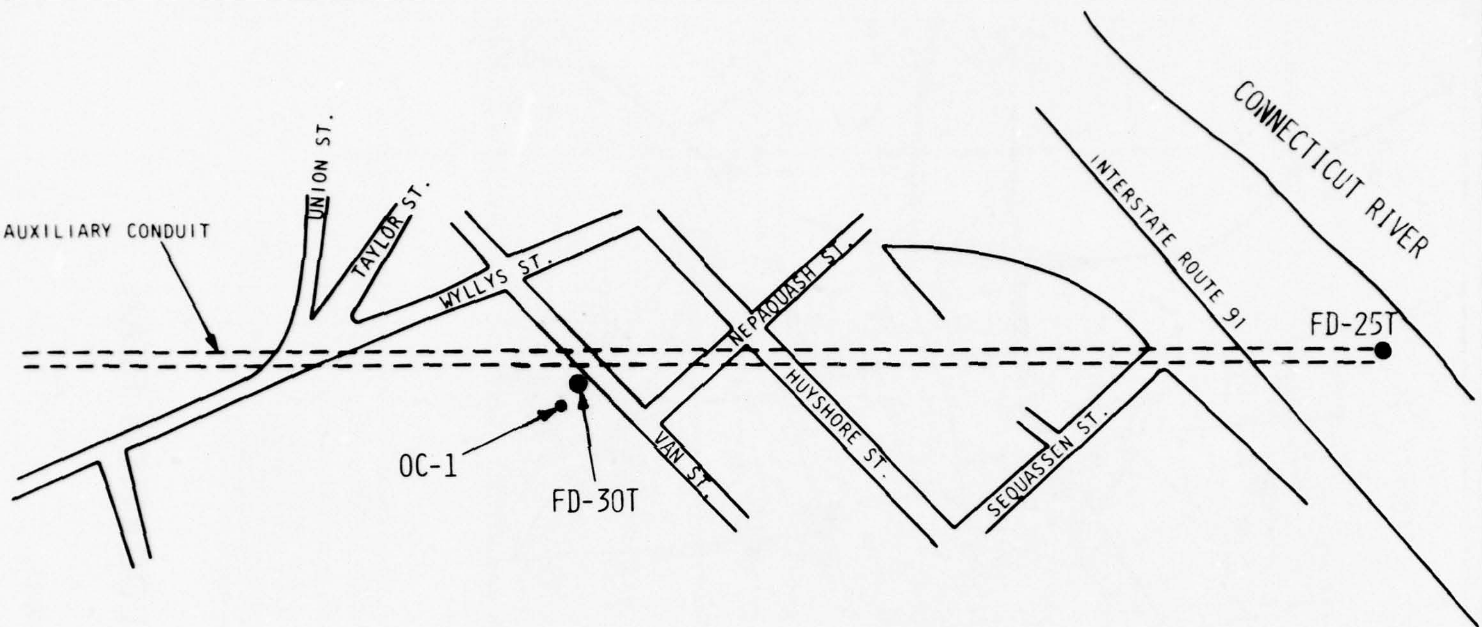
PLATE 1



IN SITU STRESS TEST LOCATION

DAMES & MOORE

PLATE 2

BORING LOG

SIZE OF CORE 5.17 INCHES DATE STARTED 5/17/77 DATE COMP. 6/30/77 GEOLOGIST T. MILLS CHECKED M. NATARAJA

COLLAR ELEV. ≈ +25' TOTAL DEPTH 155'5½ INCLIN. FROM VERTICAL 1.4 FT @ 78'10" / 1.3 FT @ 155'2" AZIMUTH DIR. OF HOLE @78'10" N 32°E / @155'2" N 16°E

TESTS PERFORMED OVERCORING WATER LEVEL DURING DRILLING AT SURFACE AFTER COMPL. 13.5' BELOW SURFACE

ROCK TYPE	DEPTH IN FEET	DESCRIPTION OF ROCK DEFECTS	RQD (75 50 25)	CORE RECOVERY (%)	IN SITU STRESS TESTS	CASING DEPTH	WATER RETURN (%) 25 50 75	OTHER TESTS
BLACK TOP OF PARKING LOT AND FILL			N/A					
	5							
BROWN FINE SANDY SILT	10							
	15							
	20							
BROWN SILTY COARSE SAND/GRAVEL	25							
BROWN SILTY MEDIUM SAND	30							
BROWN SILTY SANDY GRAVEL BROWN SILTY MEDIUM SAND	35							
RED SILT WITH VARVED CLAYS	40							
	45							
COARSE SAND, WITH GRAVEL AND COBBLES (LAYERS 2-4") IN A SILTY MATRIX (GLACIAL TILL)	50							
TOP OF ROCK →	55							
LIGHT RED TO GRAYISH WEATHERED SANDSTONE	60			100				
RED, COARSE TO MEDIUM GRAINED SANDSTONE, MODERATE TO HARD, OCCASIONALLY MICACEOUS, THIN LAYERS OF MICA CROSSBEDDED, WITH SOME THIN LAYERS OF SHALE	65	@ 67.3' TO 69.0' FRACTURE ABOUT 80° - UPPER PORTION HEALED BY MINERALIZATION		79				
@71'8" CONTACT BETWEEN RED SANDSTONE AND RED SHALE	70	@ 70.5' TO 71.1' FRACTURE ABOUT 75°, *MICACEOUS FILLING*		100 95 88				
RED SHALE, SOFT TO MODERATELY HARD, *THINBEDDED TO MASSIVE, LOCALIZED ZONES* OF CALCITE VEINS OR SEAMS; LOCALIZED ZONES OF VADOSE PISOLITES; LOCALIZED ZONES OF CLAY SEAMS	75	@76.4' TO 76.9' FRACTURED ABOUT 30° *SEMI-HEALED BY MINERALIZATION* @76.9' to 77.3' FRACTURED ZONE		99 56 100 114				Pajari @78'10"
	80							

SITE HARTFORD HOLE OC-1
IN SITU STRESS MEASUREMENTS

NOTE:

FRACTURE ANGLES REFERENCED FROM HORIZONTAL.

DAMES & MOORE

PLATE 3

BORING LOG

SIZE OF CORE 5.17 INCHES DATE STARTED 5/17/77 DATE COMP. 6/30/77 GEOLOGIST T. MILLS CHECKED M. NATARAJA

COLLAR ELEV. ≈+25 TOTAL DEPTH 155'½" INCLIN. FROM VERTICAL 1.4 FT @ 78'10" 1.3 FT @ 155'2" AZIMUTH DIR. OF HOLE @78'10" N 32°E @155'2" N 16°E

TESTS PERFORMED OVERCORING WATER LEVEL DURING DRILLING AT SURFACE AFTER COMPL. 13.5' BELOW SURFACE

ROCK TYPE	GRAPHIC LOG	DEPTH IN FEET	DESCRIPTION OF ROCK DEFECTS	RQD 75 50 25	CORE RECOVERY (%)	IN SITU STRESS TESTS	CASING DEPTH	WATER RETURN (%) 25 50 75	OTHER TESTS
			@81.8' TO 82.2' FRACTURE ABOUT 45° MINERAL STAINED.	N/A	100 100 100 100	#1 - 80'8" #2 - 80'8½" #3 - 82'9½"			
		85			89				
@89'0" TO 102'5" REDDISH BROWN SHALE, MODERATELY HARD, MASSIVE; LOCALIZED ZONES OF CALCITE SEAMS AND VADOSE PISOLITES		90	@89.0' TO 89.4' FRACTURE ABOUT 45° CALCITE SEAM, VERTICAL SLICKENSIDES		74 94 100	#4 - 92'7½"			
		95	@95.5' TO 97.1 FRACTURED ZONE @97.9' TO 98.3' FRACTURE AT 45° WITH VERTICAL SLICKENSIDES		90				
		100	@100.2' TO 101.6' FILLED FRACTURE-STARTING FROM 60° ENDING VERTICALLY, MINERALIZED WITH CALCITE		100 100	#5 - 100'6"			
@102'5" CONTACT BETWEEN RED SHALE AND DARK TO LIGHT GRAY SHALE			@102.8' TO 106.3' FRACTURED ZONE		67				
DARK GRAY SHALE, SOFT, FISSILE, LOCALIZED ZONES OF CALCITE VEINS		105	@107.0' TO 107.9' FRACTURED ZONE @109.0' TO 110.4' FRACTURED ZONE		65				
		110	@113.7' TO 114.2' FRACTURE @ ABOUT 45° @114.4' TO 114.9' FRACTURE @ ABOUT 60°, HEALED, MINERAL STAINED.		89				
@117'7" CONTACT BETWEEN GRAY SHALE AND RED SHALE		115	@116.0' TO 117.0' FRACTURE @ ABOUT 60° HEALED BY MINERALIZATION						
RED SHALE, SOFT, SUBFISSILE TO FISSLE, TO MASSIVE, LOCALIZED CALCITE SEAMS AND VEINS, IN MASSIVE ZONES			@117.6' TO 118.4' FRACTURE @ ABOUT 70° HEALED BY MINERALIZATION		89				
		120	@120.2' TO 122.1' FRACTURED ZONE		88				
			@173.6' TO 123.8' FRACTURED @ ABOUT 45°, CALCITE FILLED		100 100 100	#6 - 125'0"			
		125	@124.0' TO 124.2' FRACTURE @ ABOUT 45°, CALCITE FILLED		87				
@126'10" CONTACT BETWEEN RED SHALE AND INTERBEDDED SANDSTONE AND SHALE					85				
RED TO LIGHT REDDISH BROWN, INTER-BEDDED SANDSTONE AND SHALE, SOFT TO MODERATELY HARD, THINLY BEDDED TO MASSIVE CROSSBEDDING, WITH LOCALIZED ZONES OF CALCITE VEINS, AND VADOSE PISOLITES.		130			100 100 100 100	#7 - 132'6" #8 - 132'7"			
		135	@138.7' TO 142.1' HIGHLY FRACTURED		100 100 0 100	#9 - 132'9" #10 - 136'0"			
		140	@141.0' TO 141.8' FRACTURE @ ABOUT 75°		52				
			@143.3' TO 144.0' FRACTURE @ ABOUT 75° - HEALED BY MINERALIZATION		100				
		145	@146.4' TO 147.2' FRACTURE @ ABOUT 80°		95 100 100 86	#11 - 147'7½" #12 - 147'7½" #13 - 147'7½" #14 - 14.8'3½"			
@151'0" CONTACT BETWEEN INTERBEDDED SANDSTONE AND SHALE AND DARK GRAY SHALE		150	@151.8' TO 152.0' FRACTURE @ ABOUT 80° @152.9' TO 154.0' FRACTURED ZONE		100 66				
DARK GRAY SHALE, SLIGHTLY SILTY, MODERATELY HARD, THIN BEDDED TO MASSIVE		155	BORING TERMINATED @ 155'5½" on 6/30/77		100	#15 - 154'11½"			Pajari 115'2"
		160							

SITE HARTFORD HOLE OC-1
IN SITU STRESS MEASUREMENTS

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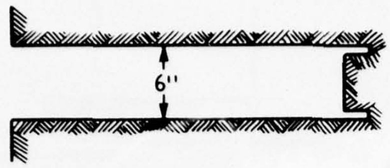
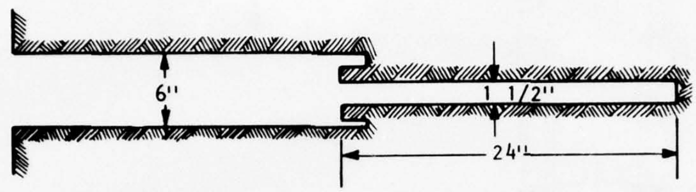
PLATE 3 (Continued)

①



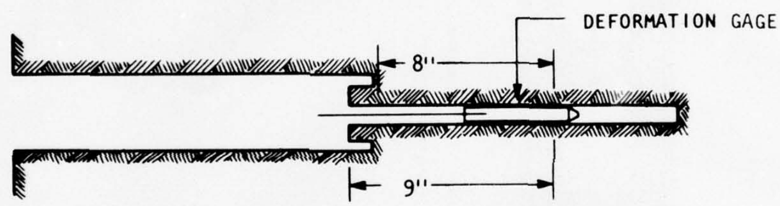
6-INCH DIAMETER HOLE IS DRILLED TO WITHIN 12 INCHES OF DEPTH AT WHICH THE STRESS MEASUREMENT IS TO BE TAKEN.

②



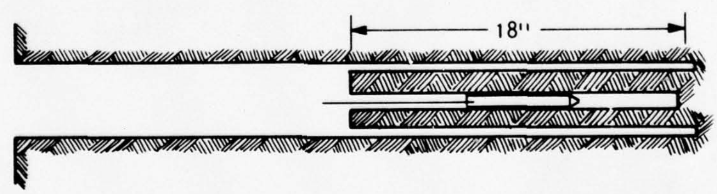
EX-BOREHOLE(1-1/2 INCH DIAMETER) IS DRILLED WITHIN THE 6-INCH BOREHOLE.

③



DEFORMATION GAGE IS ORIENTED AND POSITIONED AT DEPTH OF APPROX. 9 INCHES FROM COLLAR OF EX-BOREHOLE.

④



EX-BOREHOLE IS OVERCORED USING THE 6-INCH DIAMETER BIT FOR A DEPTH OF 18 INCHES. OVERCORE IS RECOVERED AND YOUNG'S MODULUS IS DETERMINED.

OVERCORING SCHEMATIC:

BOREHOLE DEFORMATION METHOD

(ORIENTATION MAY BE VERTICAL, HORIZONTAL , OR INCLINED)

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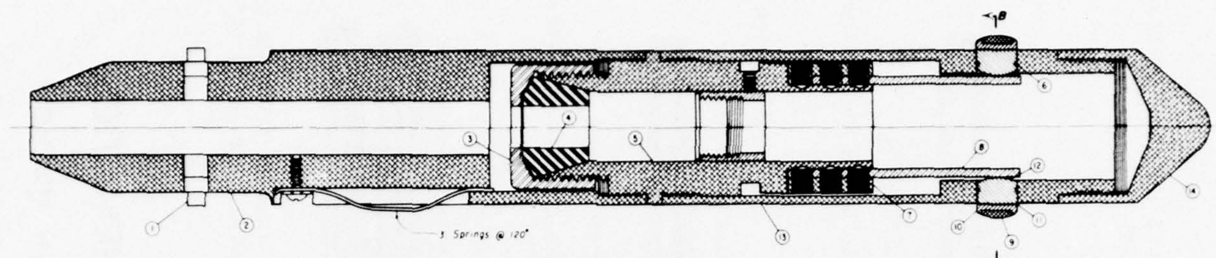
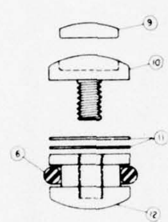
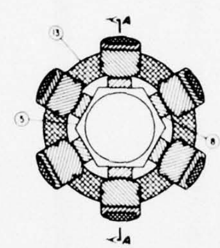


LONGITUDINAL SECTION A-A

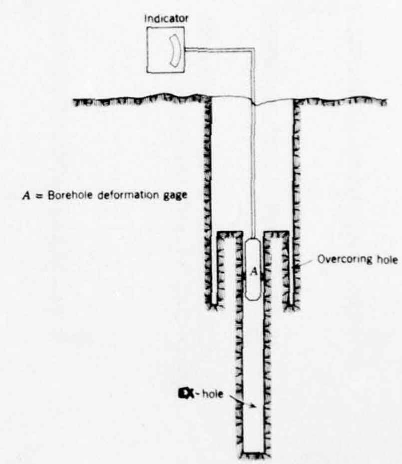
1. Lug to engage placement tool
2. Sleeve for placement tool
3. Cap for cable clamp
4. Rubber grommet
5. Body of gage
6. "O" ring seals
7. Clamp block
8. Transducer strip
9. Tungsten carbide wear button
10. Piston cap
11. Shim washers
12. Piston base
13. Case of piston
14. Cap



PISTON ASSEMBLY



SECTION B-B



SCHEMATIC VIEW
OF THE TEST SETUP

BOREHOLE DEFORMATION GAGE DETAILS

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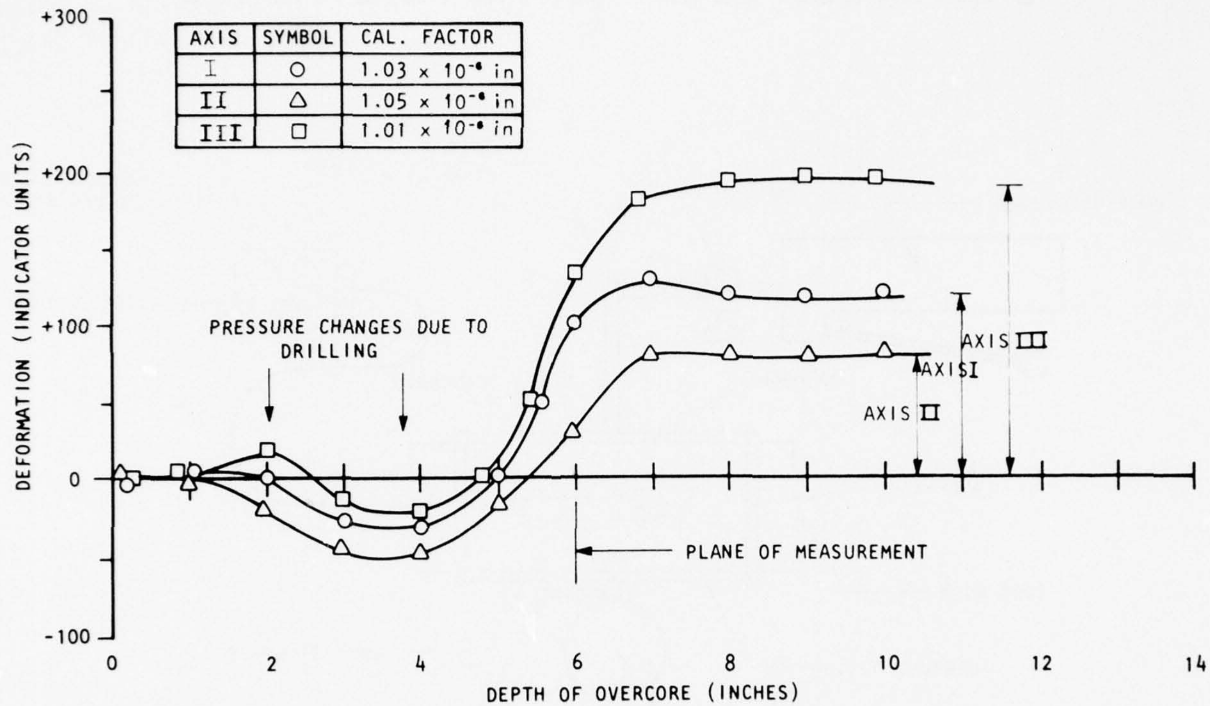
PLATE 5



IDEALIZED OVERCORE RESULTS

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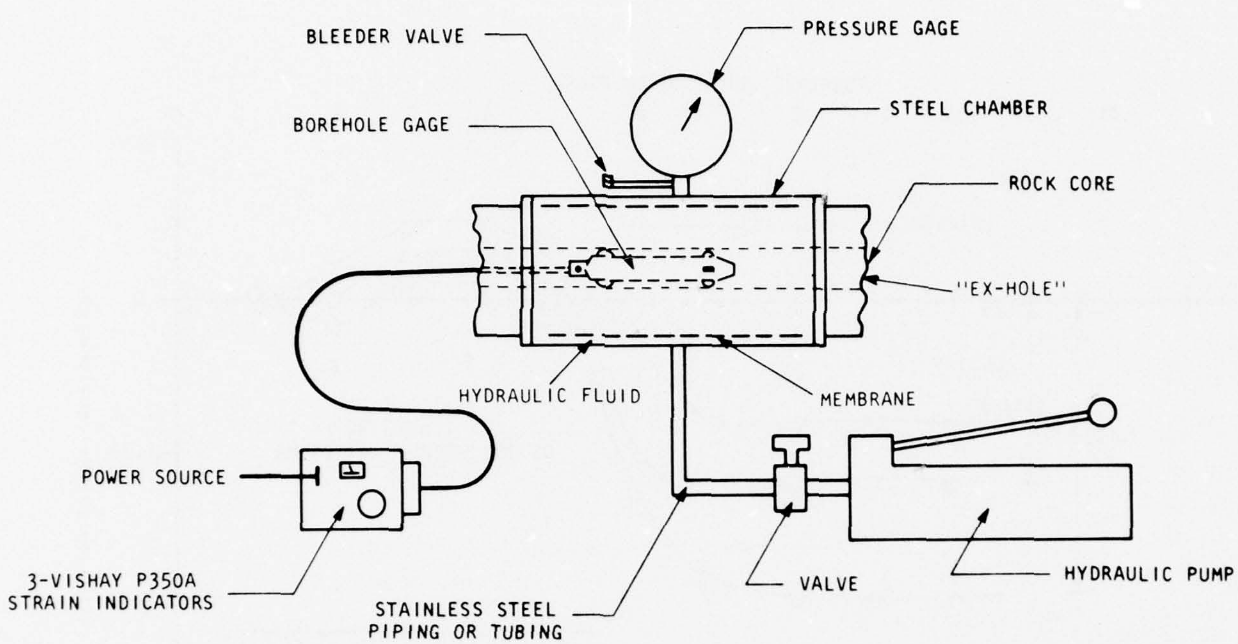
PLATE 6



SCHEMATIC : BIAXIAL TEST APPARATUS

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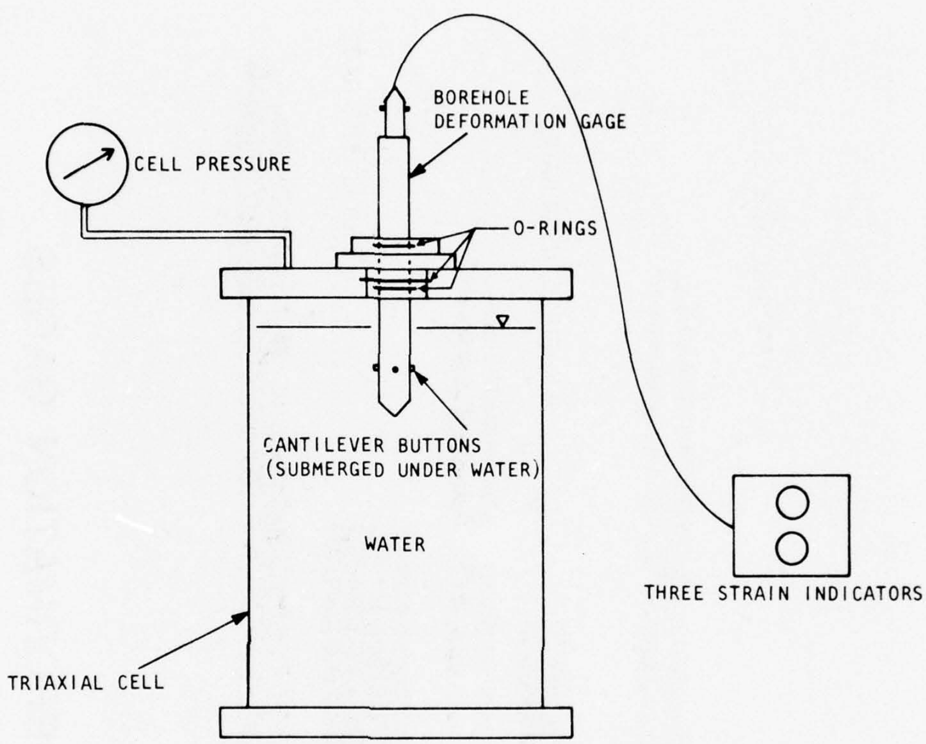
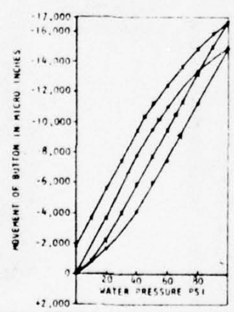
PLATE 7





TYPICAL RESULT FROM THE LABORATORY TEST

LABORATORY TEST ON STANDARD USBM BOREHOLE DEFORMATION GAGE (SCHEMATIC)

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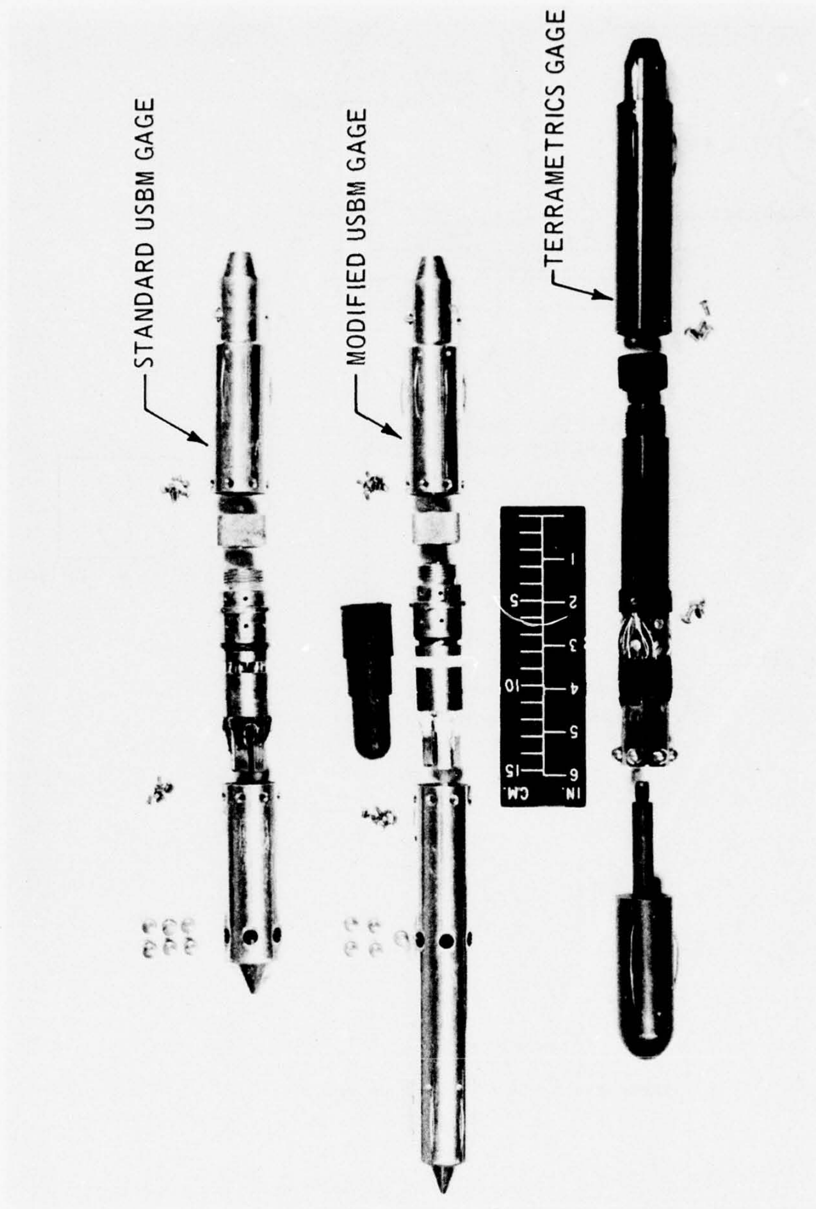
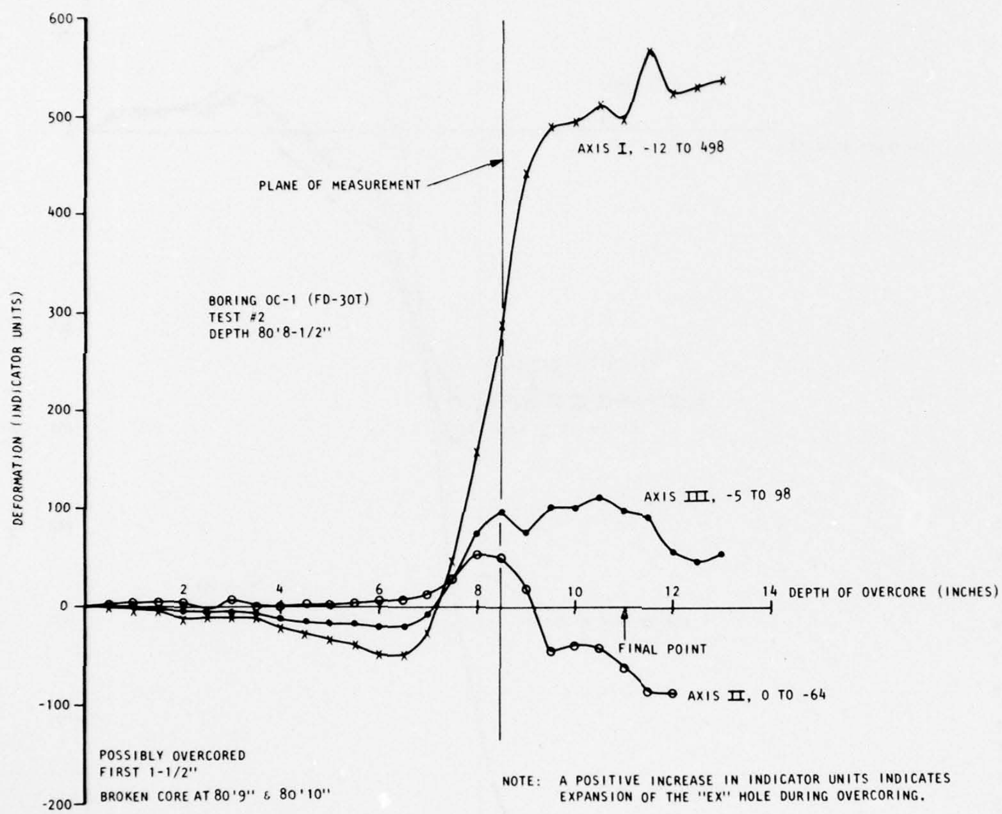


BOREHOLE DEFORMATION GAGES

PLATE 9



KEY

x AXIS I
o AXIS II
• AXIS III

K_1 = 1.09 μIN.

K_2 = 1.06 μIN.

K_3 = 1.12 μIN.

$U_1 = K_1 \times \Delta R_1 = +556$ μIN.

$U_2 = K_2 \times \Delta R_2 = -68$ μIN.

$U_3 = K_3 \times \Delta R_3 = +115$ μIN.

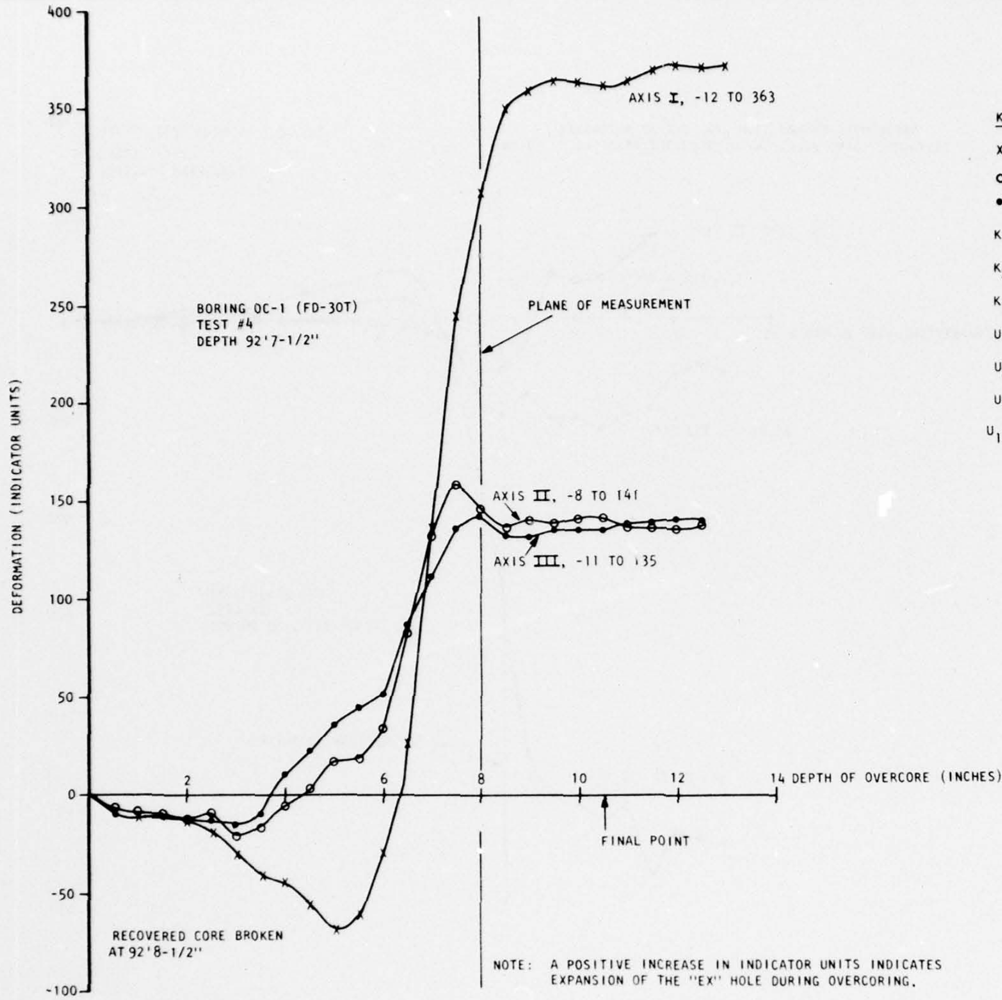
$U_{1\theta}$ = N 38° E

OVERCORE TEST RESULTS

TEST #2

DAMES & MOORE

PLATE 10



NOTE: A POSITIVE INCREASE IN INDICATOR UNITS INDICATES EXPANSION OF THE "EX" HOLE DURING OVERCORING.

KEY

x AXIS I

o AXIS II

• AXIS III

$K_1 = 1.11\ \mu IN.$

$K_2 = 1.11\ \mu IN.$

$K_3 = 1.08\ \mu IN.$

$U_1 = K_1 \times \Delta R_1 = 416\ \mu IN.$

$U_2 = K_2 \times \Delta R_2 = 165\ \mu IN.$

$U_3 = K_3 \times \Delta R_3 = 158\ \mu IN.$

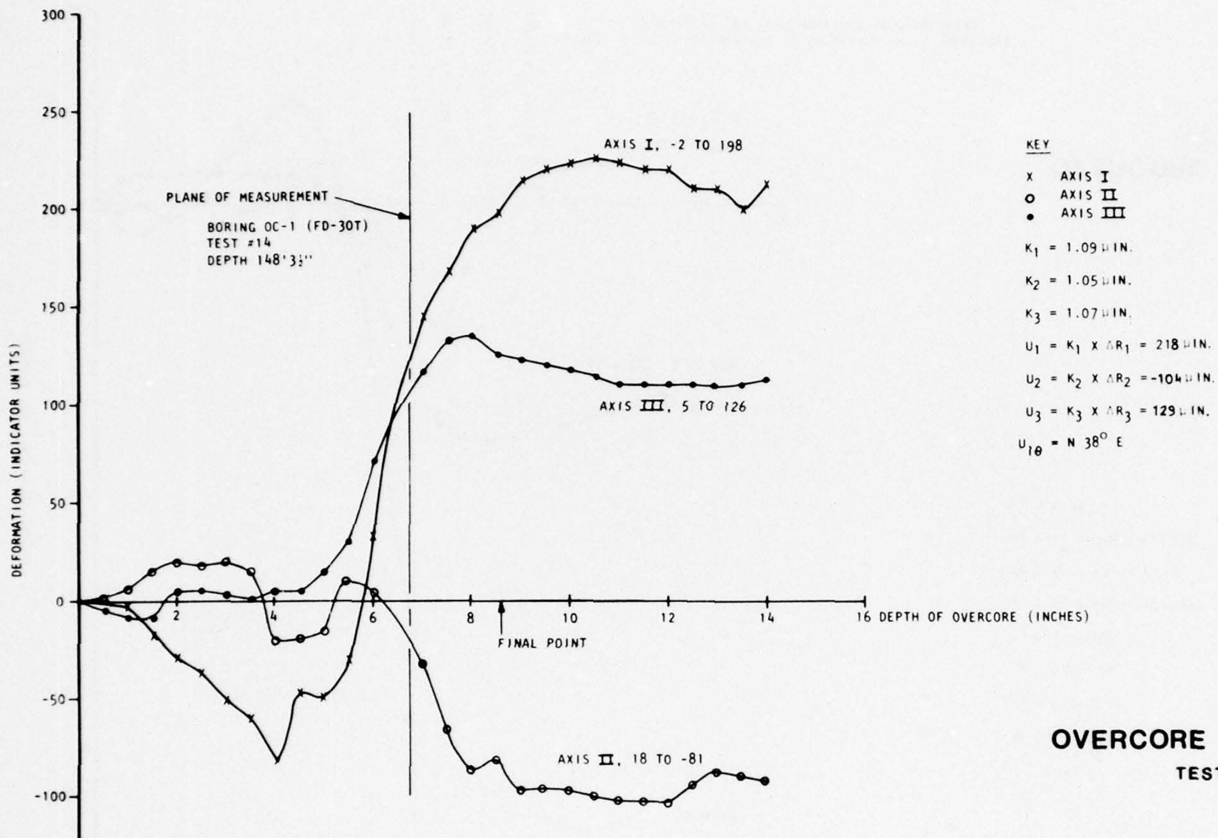
$U_{1\theta} = N\ 38^\circ\ E$

OVERCORE TEST RESULTS

TEST #4

DAMES & MOORE

PLATE 11

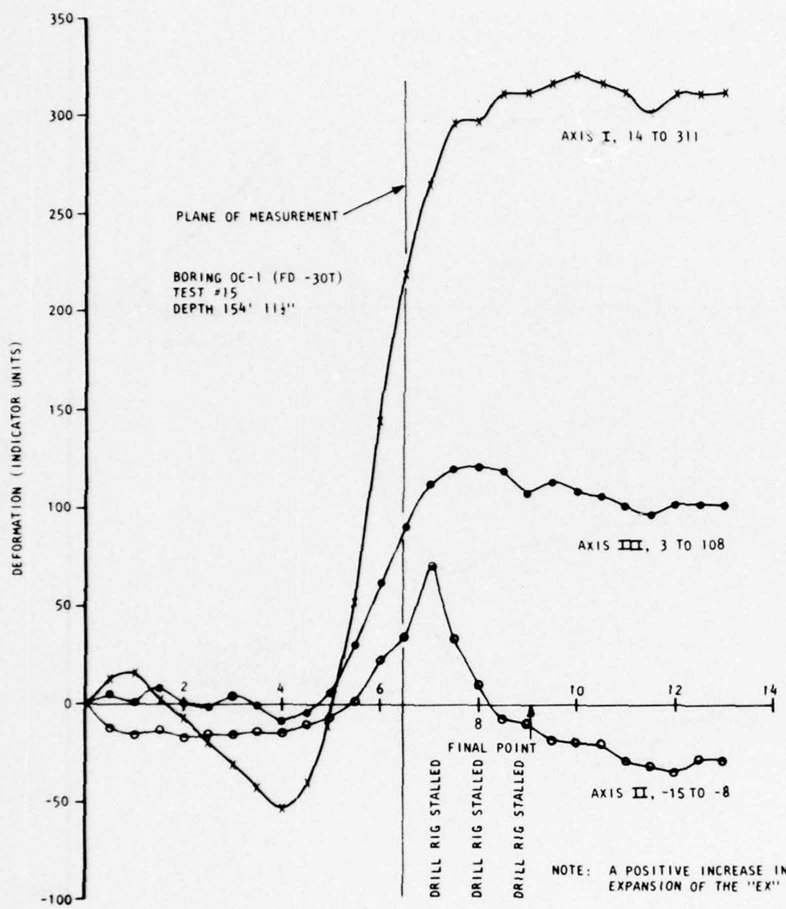


NOTE: A POSITIVE INCREASE IN INDICATOR UNITS INDICATES EXPANSION OF THE "EX" HOLE DURING OVERCORING.

OVERCORE TEST RESULTS

TEST #14

DAMES & MOORE



KEY

x AXIS I

o AXIS II

● AXIS III

K_1 = 1.08 μ IN.

K_2 = 1.05 μ IN.

K_3 = 1.08 μ IN.

$U_1 = K_1 \times \Delta R_1$ = 321 μ IN.

$U_2 = K_2 \times \Delta R_2$ = 7 μ IN.

$U_3 = K_3 \times \Delta R_3$ = 113 μ IN.

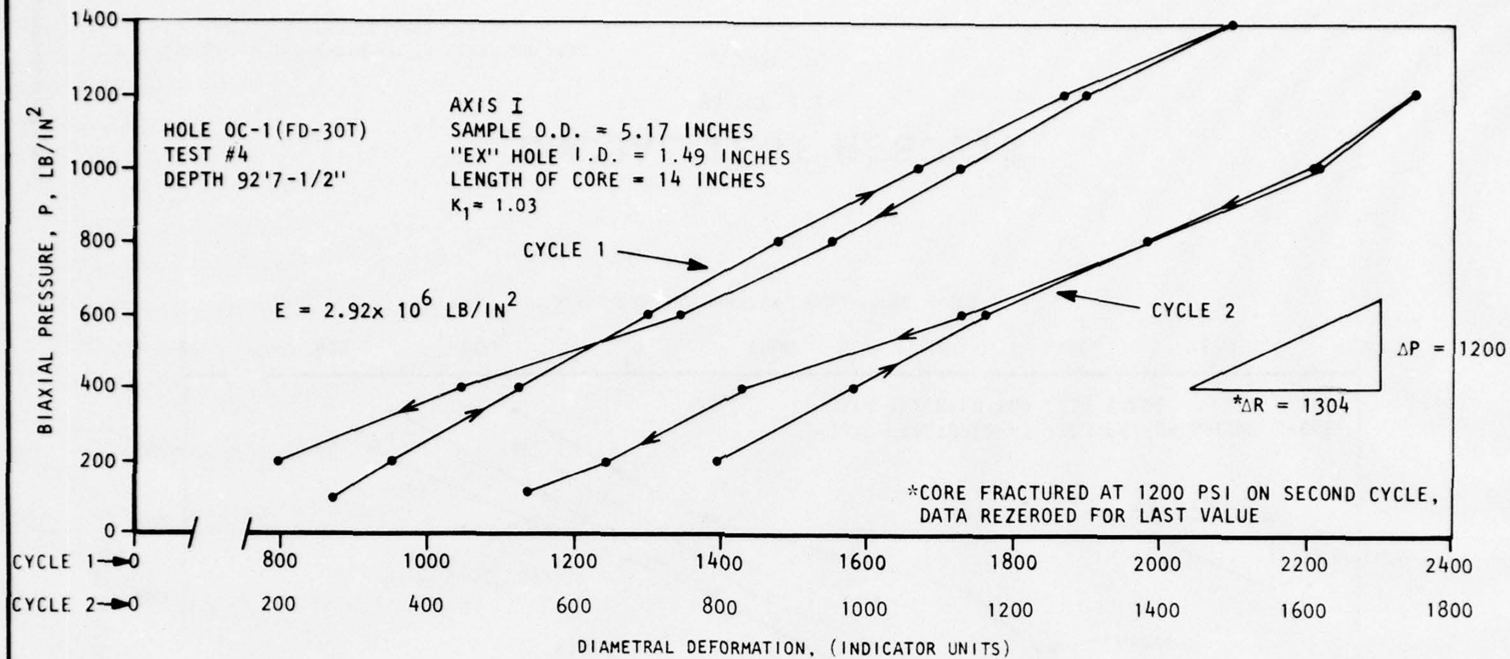
$U_{1\theta}$ = N 38° E

NOTE: A POSITIVE INCREASE IN INDICATOR UNITS INDICATES EXPANSION OF THE "EX" HOLE DURING OVERCORING.

OVERCORE TEST RESULTS

TEST #15

PLATE 13



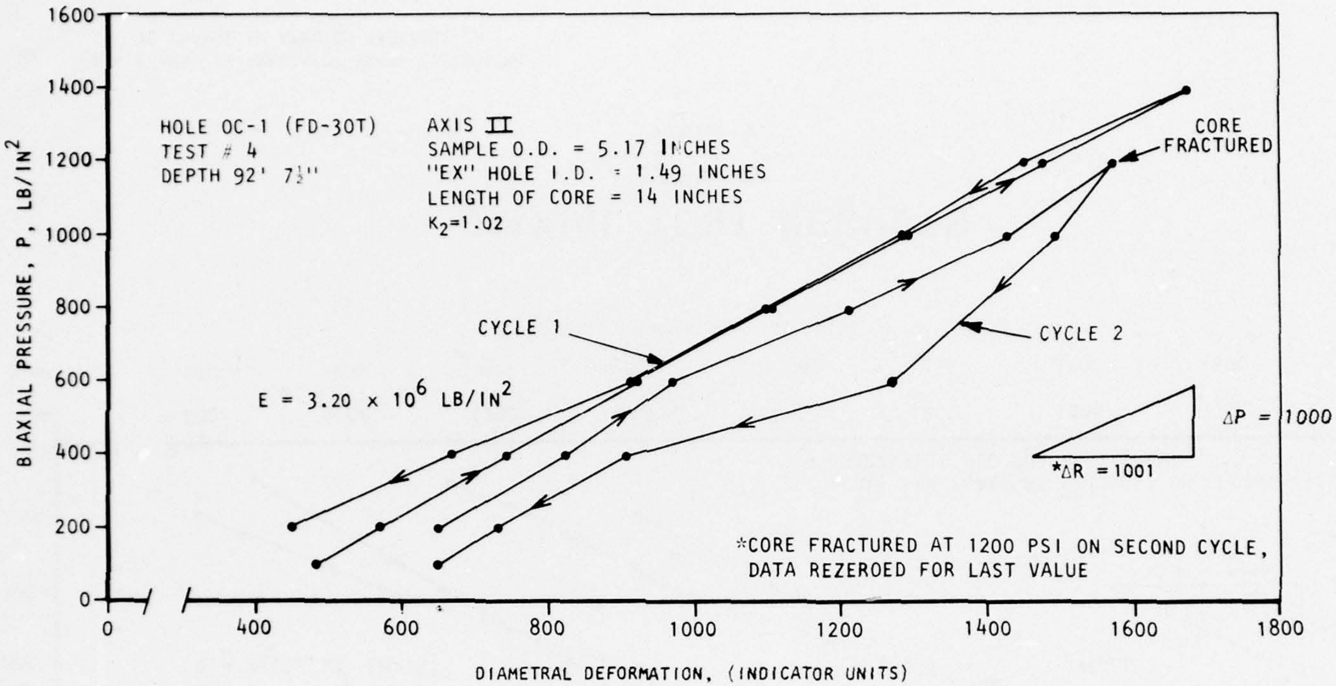
BIAXIAL TEST RESULTS

TEST #4

AXIS I

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



BIAXIAL TEST RESULTS

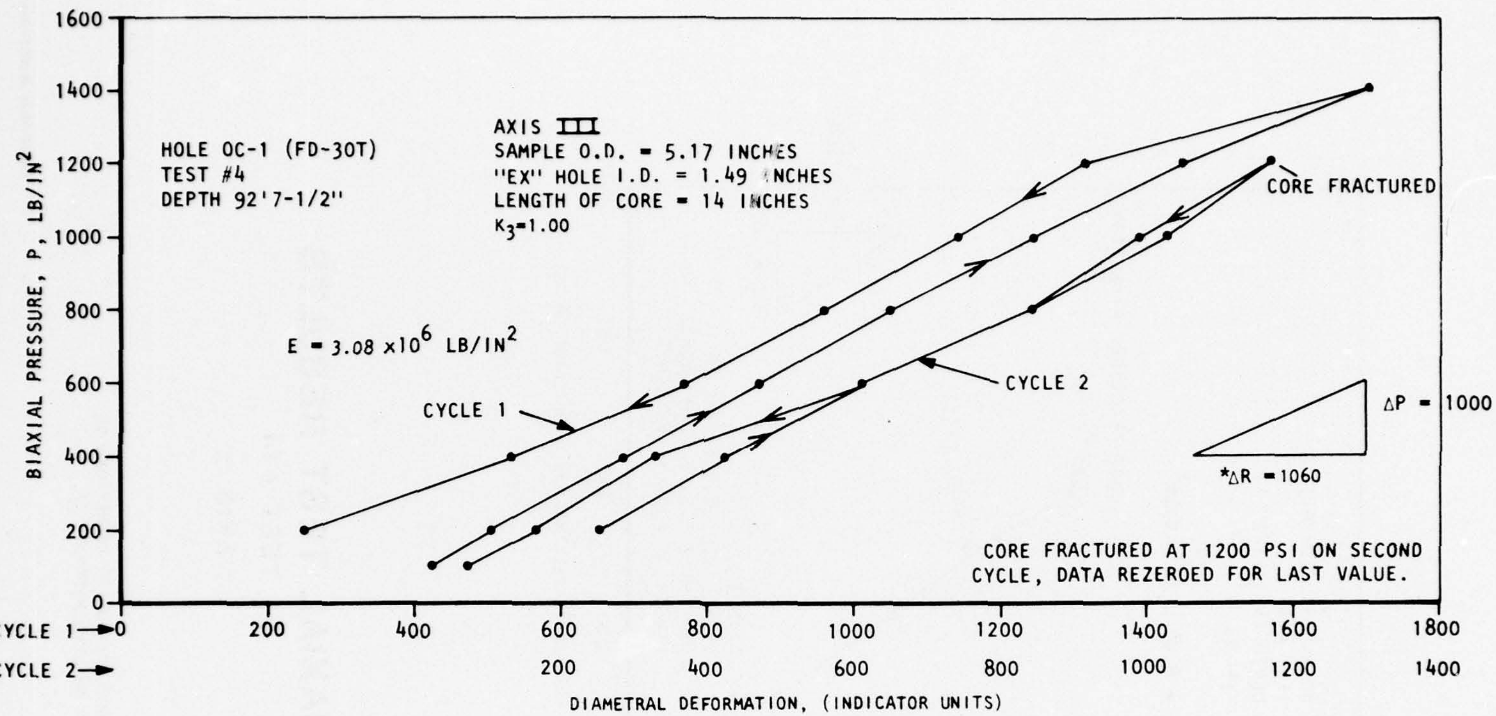
TEST #4

AXIS II

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE

PLATE 15



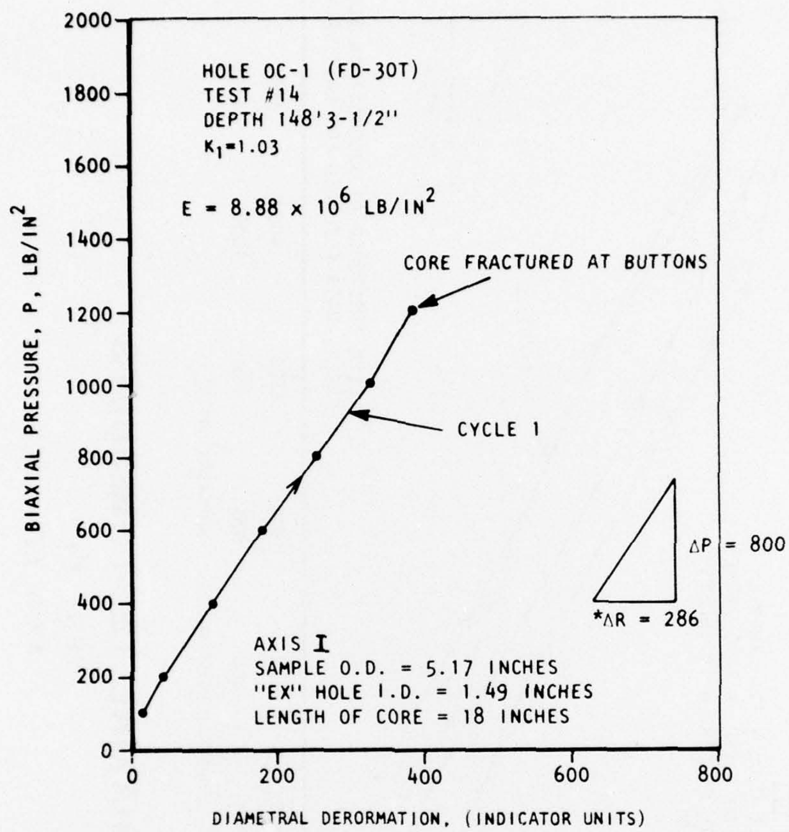
BIAXIAL TEST RESULTS

TEST #4

AXIS III

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



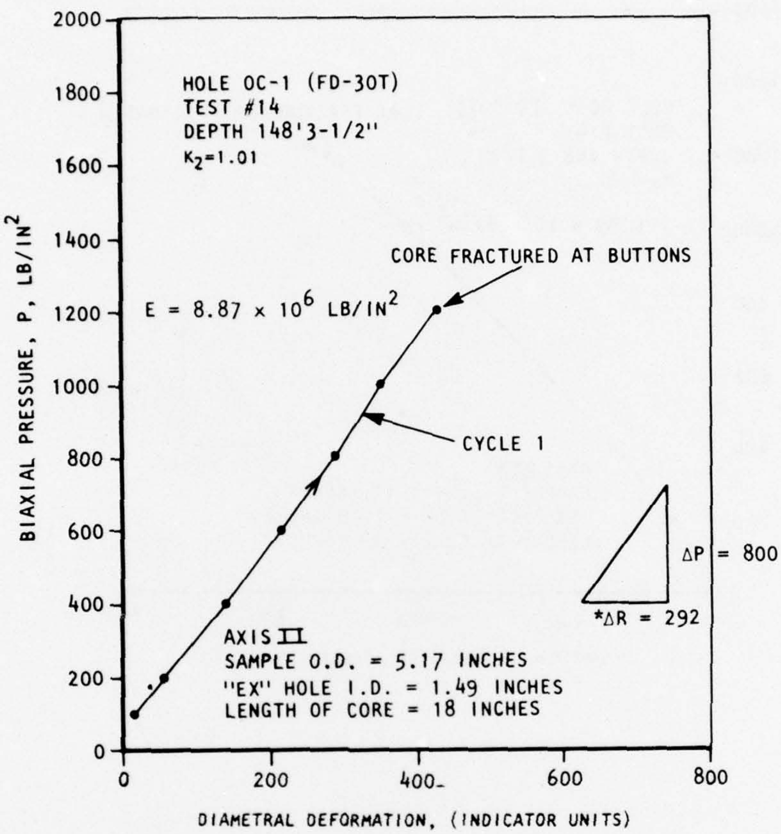
BIAXIAL TEST RESULTS

TEST #14

AXIS I

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING
TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



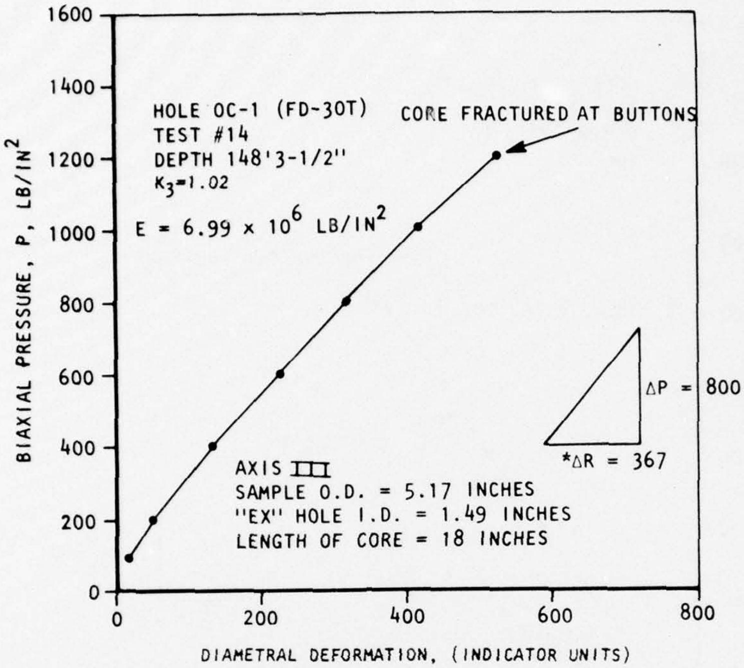
BIAXIAL TEST RESULTS

TEST #14

AXIS II

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



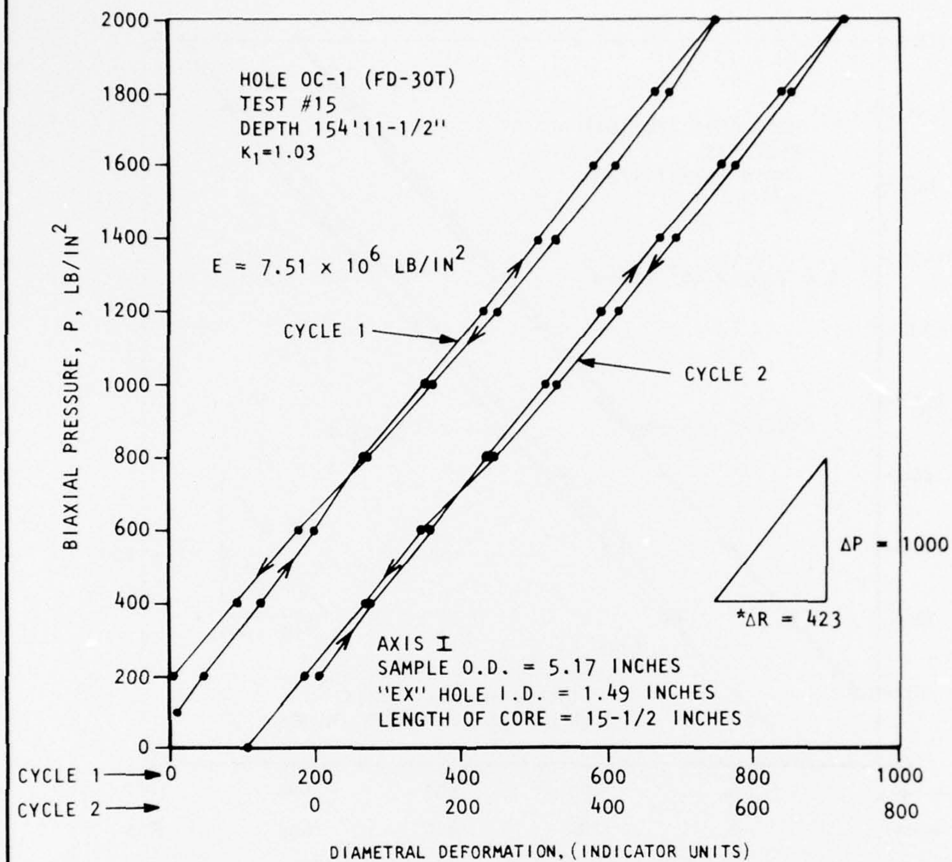
BIAXIAL TEST RESULTS

TEST #14

AXIS III

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



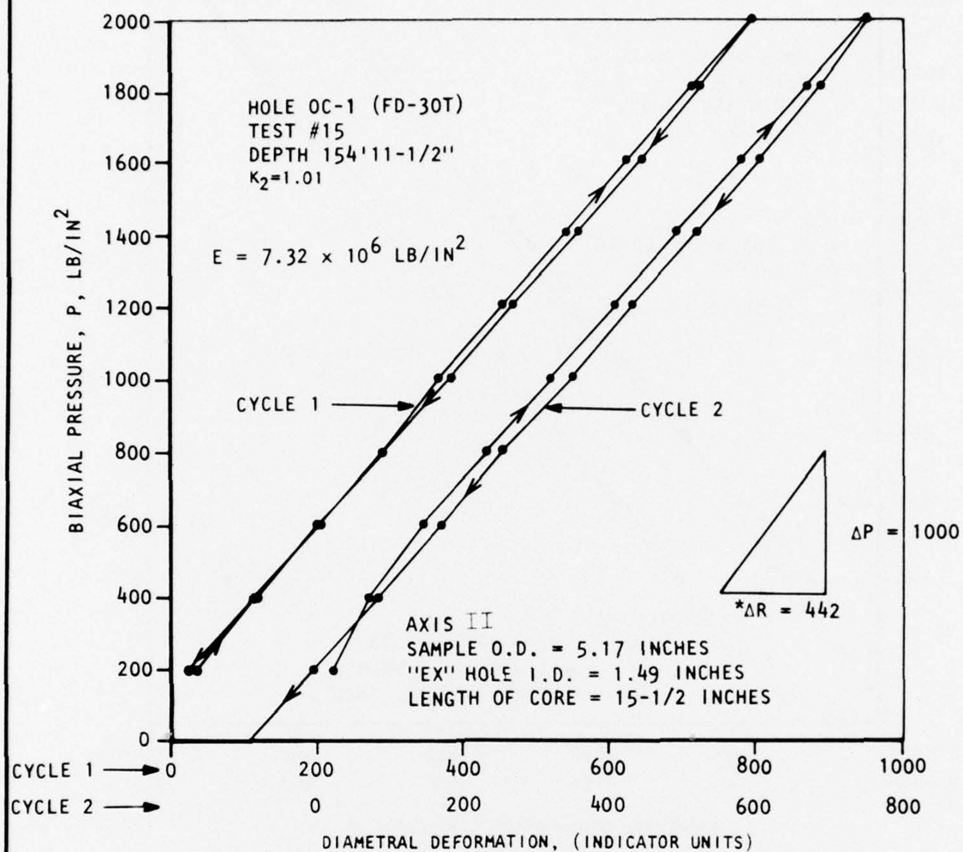
BIAXIAL TEST RESULTS

TEST #15

AXIS I

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



BIAXIAL TEST RESULTS

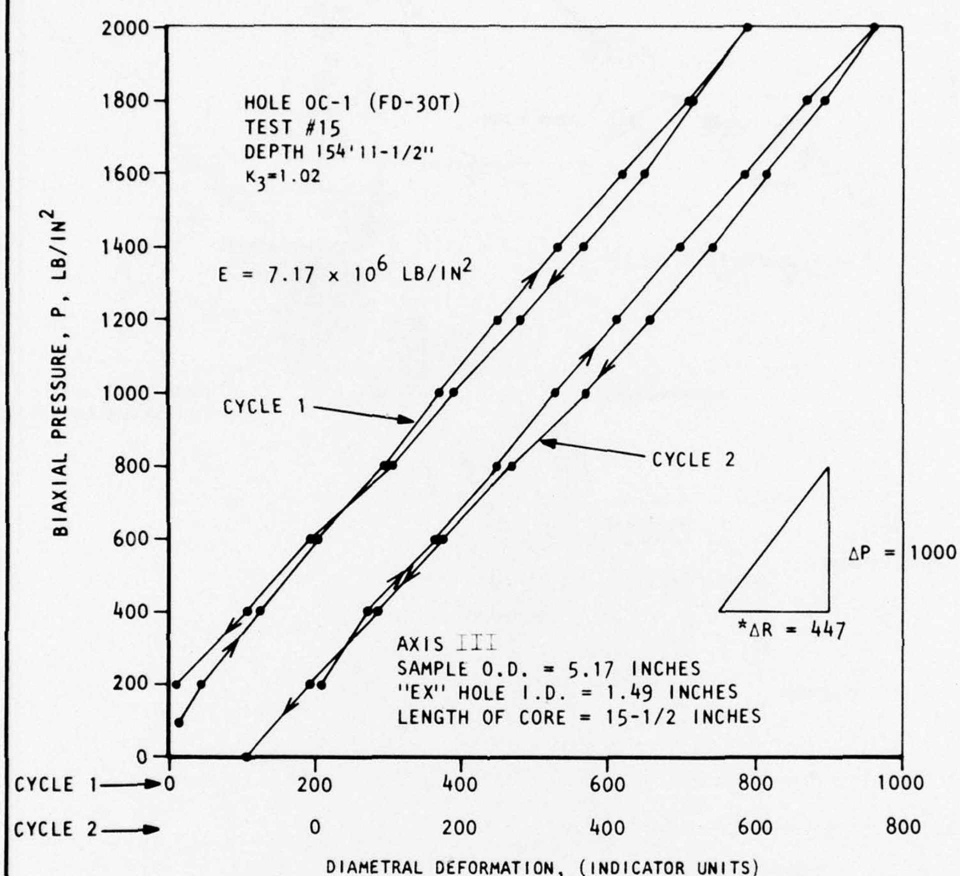
TEST #15

AXIS II

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE

PLATE 21



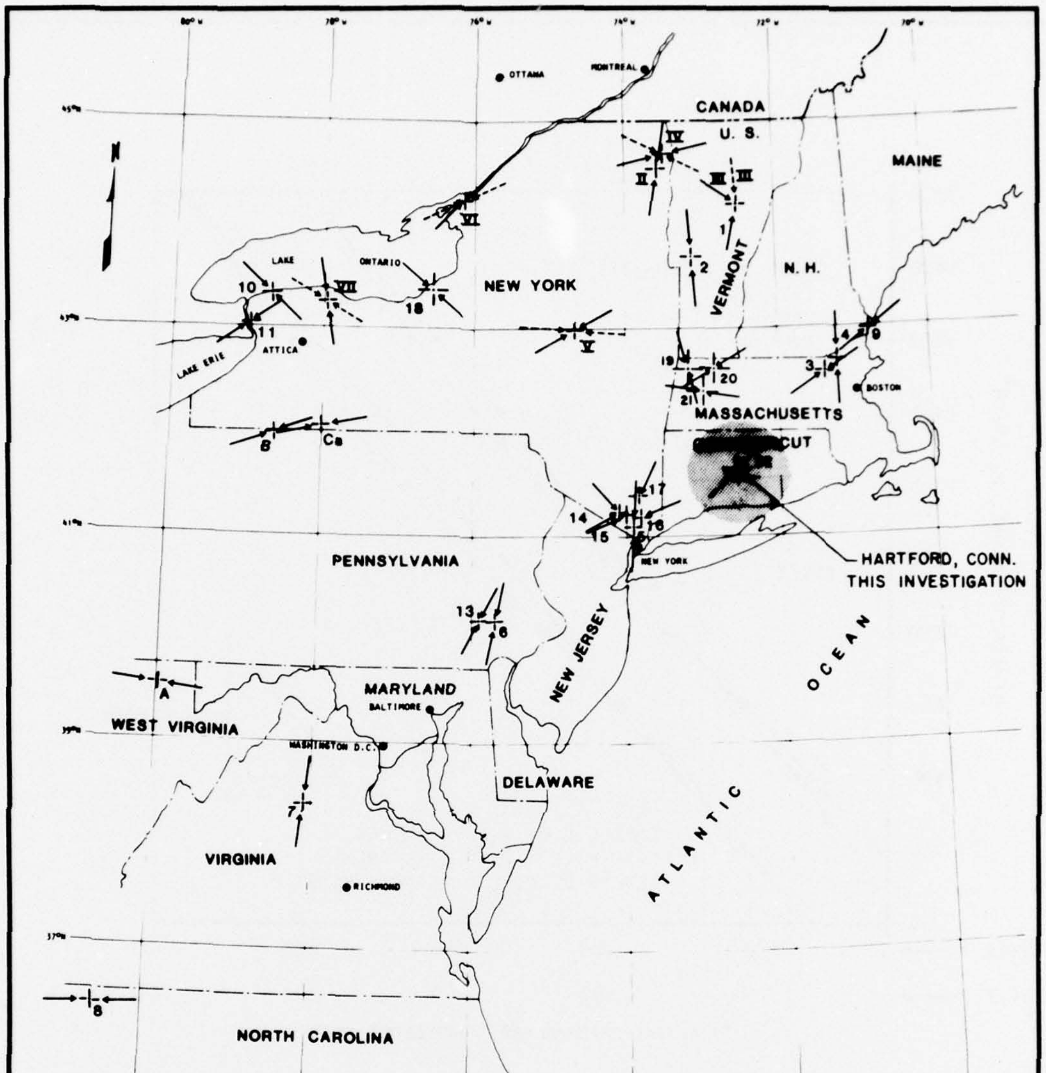
BIAXIAL TEST RESULTS

TEST #15

AXIS III

*ΔR = CHANGE IN INDICATOR UNITS CORRESPONDING TO CHANGE IN APPLIED PRESSURE ΔP.

DAMES & MOORE



IN SITU STRESS DETERMINATIONS
NORTHEASTERN U.S.

AVERAGE AZIMUTH OF HORIZONTAL COMPONENT OF MAXIMUM COMPRESSION

SCALE IN MILES: 25 0 25 50 75 100 125 150 175 200

SCALE IN KILOMETERS: 25 0 25 50 75 100 125 150 175 200

LEGEND

- TREND OF Pc
- COMPUTED TREND OF Pc
- 1 ORIENTATION OF PRINCIPAL STRESS, Pc
- A TREND OF HYDRAULIC FRACTURE, EQUIVALENT TO Pc
- I AVERAGE ORIENTATION OF MAXIMUM EXPANSION, EQUIVALENT TO Pc

PLATE 23

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Nataraja, Mysore

In situ stress measurements, Park River Project, Hartford, Connecticut / by Mysore Nataraja, Dames and Moore, Cranford, New Jersey. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1977.

31, [13] p., 23 leaves of plates : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; S-77-22)

Prepared by U. S. Department of Transportation, Washington, D. C., and U. S. Army Engineer Division, New England, Waltham, Massachusetts, under Contract No. DACW39-77-C-0037.

References: p. 30-31.

1. Borehole deformation gages. 2. Elastic analysis. 3. Isotropy. 4. Park River Project. 5. Plane stress. 6. Principal stress. 7. Residual stress measurement. 8. Stress analysis. I. Dames and Moore. II. United States. Army. Corps of Engineers. New England Division. III. United States. Dept. of Transportation. IV. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; S-77-22.

TA7.W34m no.S-77-22